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1. Titles must have a length not exceeding 50 characters and spaces.

2. A summary of approximately 50 words must accompany the paper, a 300-word synopsis must precede it, and a set of conclusions must end it.

3. The manuscript (an original ribbon copy and two duplicate copies) should be double-spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

5. Mathematics are recomposed from the copy that is submitted. Because of this, it is necessary that letters be drawn carefully, and that special symbols be properly identified. The letter symbols used should be defined where they first appear, in the illustrations or in the text, and arranged alphabetically in an Appendix.

6. Tables should be typed (an original ribbon copy and two duplicate copies) on one side of 8½-inch by 11-inch paper. Specific illustrations and explanation must be made in the text for each table.

7. Illustrations must be drawn in black ink on one side of 8½-inch by 11-inch paper. Because illustrations will be reproduced with a width of between 3-inches and 4½-inches, the lettering must be large enough to be legible at this width. Photographs should be submitted as glossy prints. Explanations and descriptions must be made within the text for each illustration.

8. The desirable average length of a paper is about 12,000 words and the absolute maximum is 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

9. Technical papers intended for publication must be written in the third person.

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Proceedings of the American Society of Civil Engineers

PENSTOCKS AND SCROLL CASES FOR NIAGARA POWER PROJECT

By J. Edgar Revelle[1] and John N. Pirok, F. ASCE[2]

SYNOPSIS

The twenty-five penstocks and scroll cases for the two Niagara Power Plants combine into a record size welded steel plate construction project. The Power Authority of the State of New York channeled water from the Niagara River above the falls to twenty-five generating units at Lewiston, New York. With a total installed capacity of 2,190,000 kw, this is now the largest hydro-electric development in the Free World. Accelerated construction schedules required completion of steel plate erection, welding and x-ray inspection on unprecedented minimum time schedules. Erection of the thirteen 24 ft 0 in. to 28 ft 6 in. increasing elbows at top of cliff by means of pin-joint bar anchors is considered unique in unconventional design and erection. Special welding procedures to produce highest feasible quality results on plates as thick as 1.812 in. had to be developed to insure minimum delays for cut-outs and repairs of welds after the x-ray inspection on all butt-welded joints.

A 50% overload strength test and complete visual examination for possible leakage was required prior to concrete encasement. A five stage construction procedure permitted rapid completion and saved approximately 2,000 tons of steel plates. Test heads were removed and rewelded to the ends of successive stage sections. Great numbers of test heads were necessary to avoid delays in testing and encasement. Each stage unit after test and inspection remained pressurized to hold roundness tolerances during concrete encasement.

Construction of the 13 penstocks in rock trenches on the 48° slope at full speed under adverse winter weather conditions presented hazardous and

difficult problems. This was further complicated by the close correlation demanded by the general contractor's wide spread rock excavation and concrete placement operations carried on day and night throughout all months of the year. As a result the combined penstock and scroll case construction project was more difficult than most previous large size welded steel plate construction undertakings.

GENERAL

The new record size Niagara Power Project at Lewiston, designed for the Power Authority of the State of New York, includes an outstanding construction project of welded steel plate work for 25 penstocks and scroll cases. The total installed generator capacity is 2,190,000 kw and exceeds the generator capacity of the Grand Coulee Hydroplant on the Columbia River. When completed, the combination of 13 generating units of 150,000 kw each (1,950,00 kw) at the Robert Moses Niagara Power Plant and 12 pump-turbine units at the Reservoir Pump Generating Plant of 20,000 kw each (240,000 kw) will be the largest hydro-electric development in the Free World. The gross head of the main plant is 314 ft. The head range for generating power at the Pump-Generator Plant ranges from 60 to 100 ft. Design data and specifications for the penstocks were prepared by consulting engineers. The construction work was carried out under their direction. A complete description of the entire Niagara Power Project was presented elsewhere.[3] Summaries of salient data and statistics are included in publications of the Power Authority of the State of New York (Monthly progress reports).

ROBERT MOSES NIAGARA POWER PLANT

The 13 penstocks for the Robert Moses Niagara Power Plant are designed for a maximum head of 405 ft including the water hammer. The diameter at the forebay is 28 ft-6 in. At the scroll case connection the diameter is 21 ft -0 in. Each penstock centerline length from forebay to scroll case connection is approximately 462 ft. Shell plate thicknesses range from 0.875 in. at the forebay to 1.812 in. at the lower elbow. Fig. 1 is a profile of the penstocks showing positions of ring supports, test heads and other pertinent detailed dimensions.

ASTM A-201-B Firebox quality steel rolled to fine grain practice was used throughout in the penstocks. Plate thicknesses are designed using a basic unit stress of 15,000 psi with 90% calculated joint efficiency for butt-welds and 1/16 in. thickness added for corrosion allowance where plates are fully stressed under operating conditions. Temporary ring girder supports are designed for a basic allowable unit stress of 24,000 psi using ASTM A7-55T structural grade steel. Welding rods used were low-hydrogen-electrodes of E7016 or E7018 type.

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[3] "Niagara Project Design," by George R. Rich and W. M. Hall, Journal, Boston Soc. of Civ. Engrs., Vol. 47, No. 1, January, 1960, p. 1.

FIG. 1.—PROFILE OF THE PENSTOCKS

FIG. 2.—TEST STAGES 3 AND 5

The usual construction procedure for erecting such penstocks would be to start at the scroll case and proceed toward the forebay. Hydrostatic over-pressure test would also be made on the completed penstock with a closure
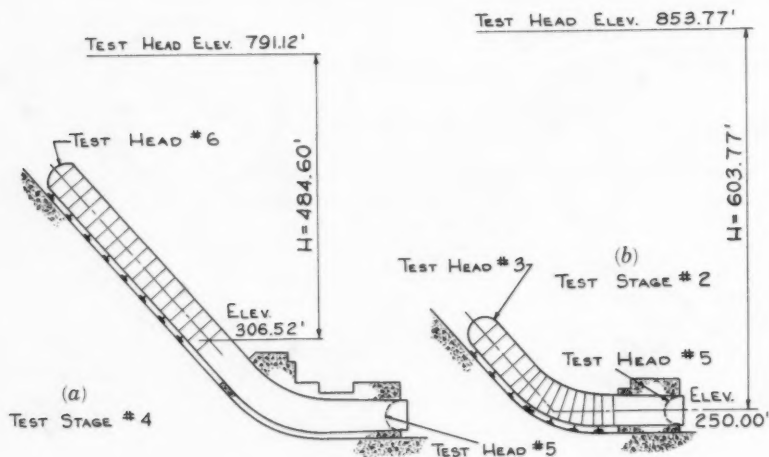

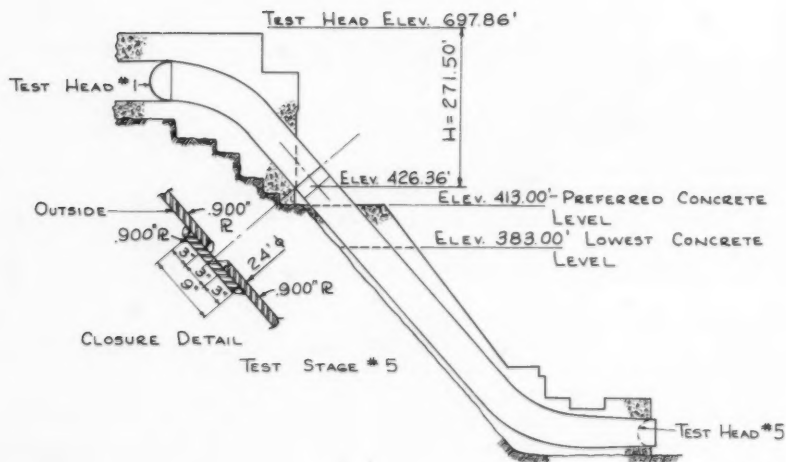FIG. 3.—TEST STAGES 2 AND 4

FIG. 4.—CLOSURE SEAM USED BETWEEN STAGES 3 AND 5

head at the forebay and another at the scroll case. On this project the con-struction calendar did not permit such simple procedure. Concrete placement had to be carried on simultaneously in the power house and the forebay sec-

tions. Consequently, the penstocks also had to be erected simultaneously at the forebay and scroll case ends. Specifications required a 50% overload strength test and visual examination of the penstocks for possible leakage prior to concrete encasement. Each penstock was divided into 5 stages.

Figs. 2, 3, and 4 show the stages of construction and hydrostatic test pressures applied to each stage. Fig. 4 also shows the closure seam used between Stages III and V.

Ring girders installed at approximately 20 ft spacing supported the penstocks during hydrostatic test and were designed to resist uplift as concrete lifts progressed. Internal pressure of 25 psi at the top of each stage was maintained to insure roundness during concrete encasement.

To fulfill the test and inspection requirement, the forebay sections of penstocks, called State III, Fig. 2(a), had to be suspended from the forebay concrete. The construction calendar again did not permit starting erection at the forebay anchorage. Construction had to start at the lower end of this elbow and proceed in reverse toward the forebay. Fig. 5 shows the temporary bootstrapping devised to accomplish this method of construction. The temporary anchorage was cut loose when the permanent anchorage shown in Fig. 6 was completed. Under full overpressure during test the Stage III penstock sections elongated elastically approximately 3/4 in. The base plates under the column supports were lubricated to reduce sliding friction forces and stresses in the column supports under such elongation attack.

The power house portions of the penstocks were broken down into Stages I, Fig. 2(b) and Stage II, Fig. 3(b). Stage I was erected, tested and encased in concrete to provide anchorage for the remainder of the penstock construction. Stage I was not pressurized during encasement. Plate thicknesses in excess of 1.7 in. were sufficiently rigid to withstand concreting pressures without distortion. When Stage II was completed the penstock projected above the power house concrete and permitted uninterrupted concreting progress in the power house area.

The gantry crane and steel trestle at power house level interfered with normal erection of State II, Fig. 3(b) and Stage IV, Fig. 3(a). A carriage on rails (Fig. 7) was employed to tuck pipe sections under the steel trestle. Stage V, Fig. 4 was erected shortly after Stage IV was tested but closure between Stage III and V was not made until encasement of Stage IV (Fig. 4) was at least two-thirds complete. The closure weld was made at night when penstock steel temperature was 65°F or less and temperature variation remained at a minimum. Welding was completed before sunrise to avoid closure weld cracks.

Before any closures were made, temperature variation studies were conducted over a period of approximately one month. Stress calculations were also made to study the effects of Poissons ratio and temperature change. The stabilizing effect by contained water indicated drastic temperature variations would not occur in pipe filled with water. Solar heat effects were also observed. The largest differential in temperature accurately measured between the sunny side and shaded side was 45°F. One estimated condition was 50°F differential but the recorder overran the chart, consequently the data is not accurate. A 30°F to 42°F differential was common at various ambient temperatures. The maximum difference did not occur at a maximum ambient temperature. Recording thermometers were used to obtain the data. Fig. 8 shows the thermometer locations and Table 1 shows the record for two days of observation.
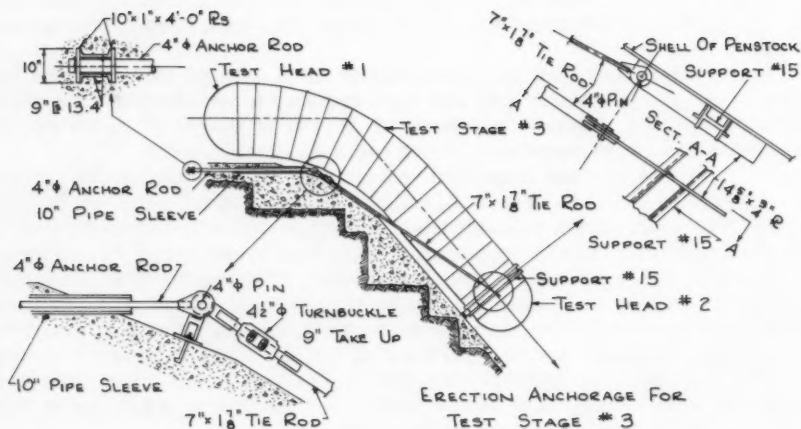
FIG. 5.—TEMPORARY BOOT-STRAPPING
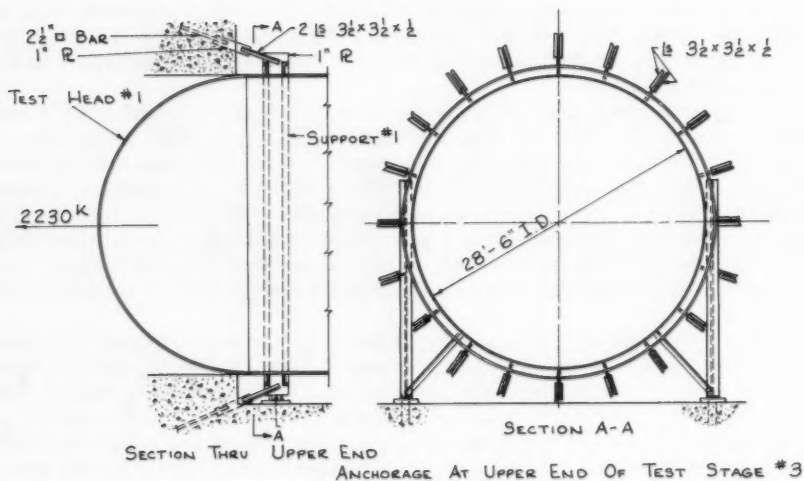
FIG. 6.—PERMANENT ANCHORAGE

Testing in five stages on the 13 penstocks was an innovation which facilitated placement of the concrete (at high speed schedules) in the large volume areas at the intake gate and power plant levels. A single test on an entirely completed penstock with a specified test pressure of 262 psi for Stage I would have required greater plate thicknesses in the upper sections to safely withstand this test pressure. Approximately 2000 tons of steel plate were saved by
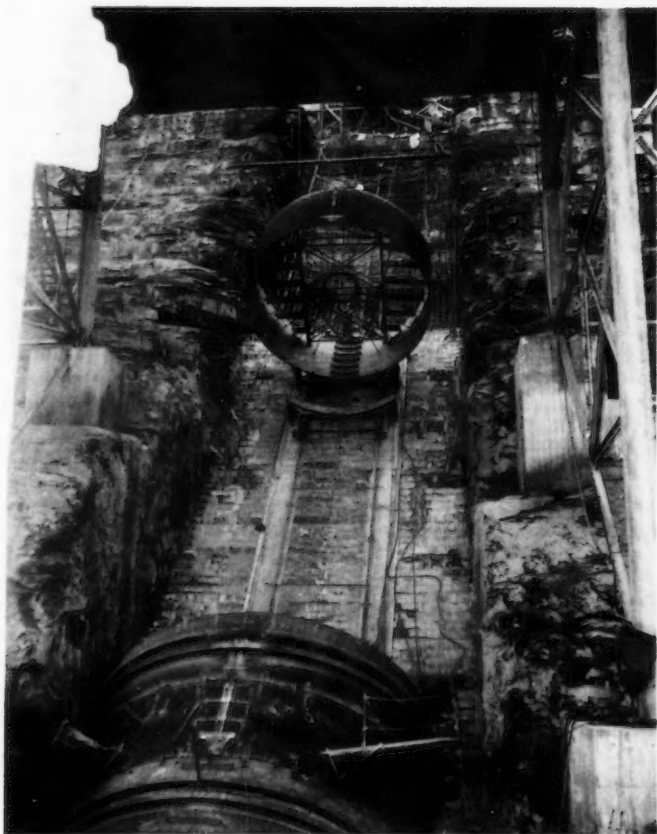
FIG. 7.—TRANSFER CAR

using the stage test method. The hydrostatic test pressures for the various stages were specified in feet of water and were measured as follows:

Stage III, Fig. 2(a) - Upper Elbow 264.0 ft above elevation 433.85 ft.
Stage I, Fig. 2(b) - Lower section connection to scroll cases 603.77 ft above centerline of test head No. 5 elevation 250.00 ft.

TABLE 1.—TEMPERATURE MEASUREMENTS

Time	Thermometer Location					
	1	2	3	4	5	6
(a) October 4, 1960						
6:00 AM	47°	60°	56°	65°	67°	58°
8:00 AM	46°	60°	56°	64°	66°	59°
10:00 AM	47°	60°	55°	64°	66°	60°
12:00 N	50°	60°	56°	64°	66°	62°
2:00 PM	72°	70°	58°	66°	66°	64°
4:00 PM	96°	79°	61°	66°	67°	66°
6:00 PM	105°	70°	63°	66°	68°	66°
8:00 PM	74°	62°	64°	66°	68°	63°
10:00 PM	57°	61°	62°	66°	68°	60°
12:00 Mid	50°	60°	60°	65°	67°	58°
(b) October 9, 1960						
6:00 AM	44°	58°	53°	63°	65°	46°
8:00 AM	44°	58°	52°	63°	65°	46°
10:00 AM	47°	59°	52°	64°	65°	53°
12:00 N	54°	67°	56°	64°	65°	57°
2:00 PM	78°	78°	60°	66°	66°	60°
4:00 PM	96°	79°	64°	66°	66°	63°
6:00 PM	107°	63°	67°	67°	66°	64°
8:00 PM	72°	61°	66°	66°	66°	58°
10:00 PM	58°	60°	63°	66°	66°	55°
12:00 Mid	52°	60°	61°	65°	66°	42°
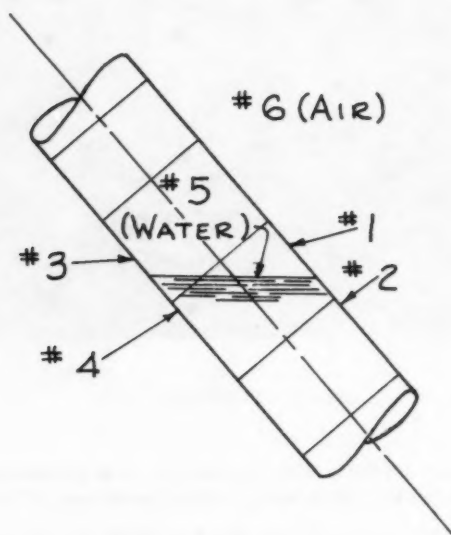
FIG. 8.—THERMOMETER LOCATIONS

Stage II, Fig. 3(b) - Lower Elbow Same pressure as Stage I
Stage IV, Fig. 3(a) - Long sloping section 484.60 ft above elevation 306.52 ft.
Stage V, Fig. 4   - Closure Ring 271.50 ft above elevation 426.36 ft.

Stage numbers are listed in the order of actual construction sequences.

Using unheated water at temperatures approximating the river water added to the severity of the testing of the 13 penstocks for the Robert Moses Niagara Generating Plant. Possibilities of cracks developing in welds on plates at low temperature in the operational ranges were practically eliminated by the special test requirements. Careful visual inspections of all outside weld surfaces of the 13 penstocks were made during testing. Test head No. 5 at the scroll case connection and test head No. 1 at the forebay remained in place until the final tests were completed on the scroll cases and the Stage V closure rings. The multiple use of test heads Nos. 2, 3, and 4 by removal with air-arc-gouging and rewelding on other penstock stage sections was limited because of the accelerated scheduling.

High quality workmanship and welding requirements supplemented the general standards of the Section VIII of the ASME code for Unfired Pressure Vessels (1956). The consulting engineers for the Power Authority of the State of New York, specified additional technical provisions commensurate with the functions of the penstocks and scroll cases as components on this important power project.

Descaling.—All mill scale was removed by the phosphoric acid pickling process and facilitated inspections for surface or edge defects, such as shear cracks or laminations. Plates with manholes or other sizable connections were stress relieved in accordance with Code paragraphs UW-40 and UCS-56 and the attachment welds were subsequently magnafluxed.

Tolerances.—Tolerances for out-or-roundness of cylindrical sections conformed with paragraph UG-30 of the Code, with supplemental requirements: (a) The difference between maximum and minimum diameters at any cross-section had to be less than 3/4 in. (b) The permissible offset between plates at butt-joints, either circumferential or longitudinal, could not exceed 1/16 in. for plates up to and including 1 in. thickness, or 1/8 in. for plates over 1 in. thickness.

Welding.—Precautions against notches and stress raiser conditions were extensive. All temporary welds for erection attachments, all burrs and weld spatter were carefully removed by either air-arc-gouging, chipping, grinding, or brushing so as to leave clean surfaces on the plates. Undercuts, pits and gouges were filled by welding then chipped smooth to preclude reductions in the design thicknesses. Tolerances for smoothness of all inside plate surfaces were the same as specified for butt-welds.

Manual welds were carefully back-grooved by air-arc-gouging in order to completely remove slag or defects or cracks that might have developed in the root pass. Magnafluxing was used to check the results.

Automatic welds for plates over 1-1/2 in. thickness were required to be multi-pass on each side. For plates 1-1/4 in. or less in thickness, single pass (one pass from each side) welding was not allowed unless the joint design and welding procedure were first approved as insuring 100% fusion without back-grooving.

Preheating.—Preheating at welds was in accordance with recommendations of paragraph UW-30 of ASME Code (1956) and as necessary to preclude

cracks or defects in welding. Preheating was done with propane gas using multiple jet burners. During winter months the welding operations were protected with plastic sheeting enclosures.

Radiographic Inspection.—Complete x-ray inspection of all butt-welds in plates was required to comply with Code paragraph UW-51. The Gamma Ray method was not permitted. A supplemental quality provision specified that the weld ripples on weld surface irregularities on both the inside and outside, were to be removed to a degree such that the resulting radiographic contrast due to any remaining irregulatities could not mask or be confused with that of any objectionable defect. The penstock contractor maintained a completely equipped dark room developing laboratory with viewing equipment to permit the engineer inspector to review and evaluate the films within 24 hr after exposure. Filing of all films at the site and developing and viewing facilities permitted immediate review and comparisons of retest films. The x-ray films were thoroughly utilized for continuous quality control of welding operators, equipment, wire and procedures. Prompt x-raying followed the progressive depositing of weld metal so as to insure against any deviations from high quality standards. The testing and encasement dates often were so close to welding completions that the normal time based on usual averages for weld repairs and retest x-rays could not be tolerated. Therefore, very special controls were necessary to insure exceptionally high quality welding at the accelerated schedules. In addition to the 100% x-ray inspection of all the butt-welds in shell plates, spot x-rays and magnafluxing as required were used for inspection and quality control of the welds on the structural sections of the ring girders for 13 penstocks. Cracks in these members could be propagated into the shell plates if precautions were not maintained to prevent their occurrence.

Scroll Cases.—The 13 turbines in the Robert Moses Power Plant were furnished by two companies. Thirteen scroll cases were erected by the penstock contractor. Six of these scrolls cases were also fabricated by this firm. The scroll case plates were fitted to the stay rings in the shop and shipped knock-down to the job site for reassembly and welding to the stay rings by the penstock contractor. Fig. 9 shows shop fitting of the scroll case plates to the stay rings.

The scroll cases are Carilloy T-1 (Firebox quality with maximum cross rolling and in accordance with ASME Code 1204-3) steel. Welding and workmanship conformed with the general standards of Section VIII of the ASME code for Unfired Pressure Vessels (1956). Plate thicknesses varied from 1-1/16 in. at the inlet to 3/8 in. at the terminus of the scroll. Inlet diameter is 21 ft -0 in. Preheating was used for all welds. All butt welds in the T-1 scroll cases were magnafluxed. The butt-weld between the scroll case plates and the stay rings were fully radiographed. All intersections were x-rayed. The balance of the butt-welds between scroll case plates were random radiographed. All erection assembly gadgets (inside and outside) attached to the T-1 plates were also of T-1 material to prevent creating spots of weaker materials by welding dilution. All surfaces inside and out were cleared smooth of all burrs, scars and undercuts prior to hydrostatic test. All internal bracing provided for assembly was also removed prior to test. These precautions were taken to eliminate as far as possible all crack starters and stress raisers in the finished structure. Fig. 10 shows the exterior of the scroll cases and Fig. 11 shows the interior of the cases. All permanent attachments such as brackets and anchors are of
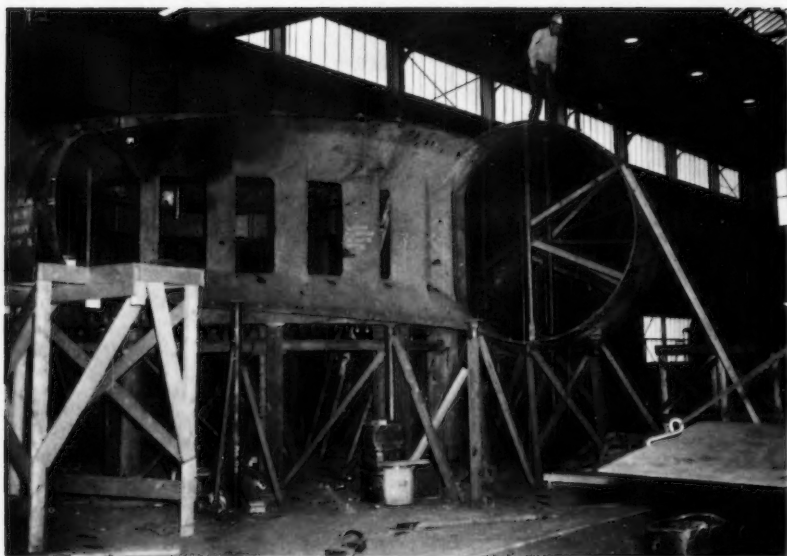
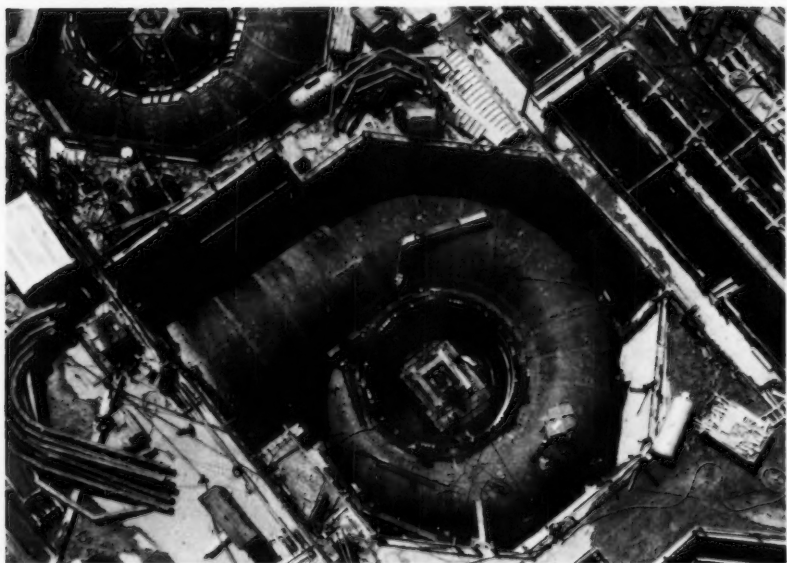FIG. 9.—SHOP FIT-UP T-1 SCROLL CASES

FIG. 10.—EXTERIOR VIEW OF T-1 SCROLL CASES
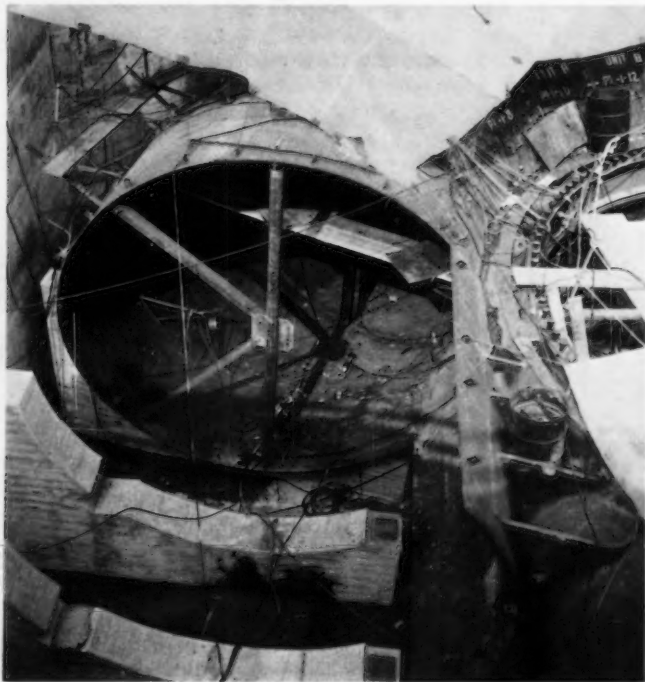
FIG. 11.—INTERIOR VIEW OF T-1 SCROLL CASES

FIG. 12.—CONCRETE SADDLES T-1 SCROLL CASES

T-1 material so as to be compatible strain and strength-wise when the T-1 shells are fully loaded.

The scroll cases were supported in concrete saddles during test (Fig. 12). Each scroll case was hydrostatically tested to 150% of maximum working head. Actual test pressure was 275 psi. The scroll cases were pressurized during concrete encasement. Water provided ballast to resist concreting uplift and internal pressure prevented distortion by external concreting pressures.

The scroll case erection and welding was carried on simultaneously with the penstock erection. To minimize the fitting time in the crowded power house section, segments of the scroll cases were fitted in one of the two assembly yards maintained by the penstock contractor. Sections were accurately braced and positioned and hauled as a unit to final location.

RESERVOIR PUMP - GENERATING PLANT

The pump generating plant is a relatively low head unit that required moderate thickness steel in the penstocks and scroll cases. The 12 penstocks taper from 24 ft - 0 in. at the forebay to 18 ft - 0 in. at the scroll case connections. Plate thicknesses ranged from 0.47 in. at rhe forebay to 0.687 in. at the scroll cases. Centerline length of each penstock is 92 ft. Fig. 13 and Fig. 14 show general details of the pump generating plant and penstock.

Penstocks.—ASTM-A201 Firebox quality steel to fine grain practice was used throughout in the penstocks. The stiffening rings, supports and anchors were ASTM A-7 steel. Only one plate in each penstock containing a 22 in. diameter drain nozzle was stress relieved. Welding at this nozzle was magnafluxed before and after stress relieving.

Less rigid test requirements were specified for the 12 penstocks with generating and pumping heads ranging approximately between 60 ft to 100 ft. Buttwelds in the plate thicknesses extending from 0.47 in to 0.68 in. were inspected by spot x-raying in accordance with American Water Works Association Specification, AWWA D-100-55. Supplemental, specifications included the same additional requirements for on site x-ray film developing and viewing facilities that applied to the 13 high head penstocks. Vacuum box testing of all butt-welds to detect leakage was specified, but hydrostatic pressure tests on the 12 penstocks with all plates less than 3/4 in. thickness were not required.

The penstock contract included providing adequate internal stiffener spiders as required to maintain roundness of the 12 penstocks and all temporary supports and hold down anchors to prevent flotations or misalignment during pouring of encasement concrete.

Fig. 14 shows the supports and penstock bracing.

The engineering factors leading to the decision that the hydrostatic testing was not warranted for the 12 penstocks at the pump turbine plate were:

1. Plate stresses were low and were all within ductile fracture limits for the specified fine grain steel.

2. Rigid controls insured high quality welds.

3. Operating heads were low and penstock lengths were short in comparison with 13 penstocks at the large power plant.

Scroll Cases.—The steel plates used for the 12 scroll cases were ASTM A-285-C Fire Box quality steel.
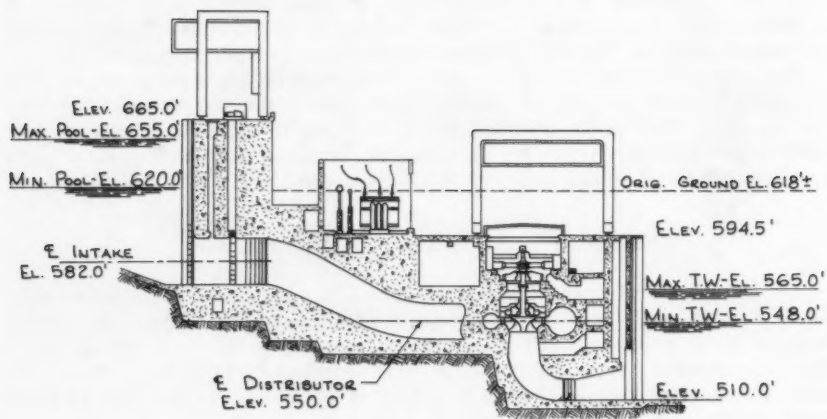
FIG. 13.—PUMP GENERATING PLANT
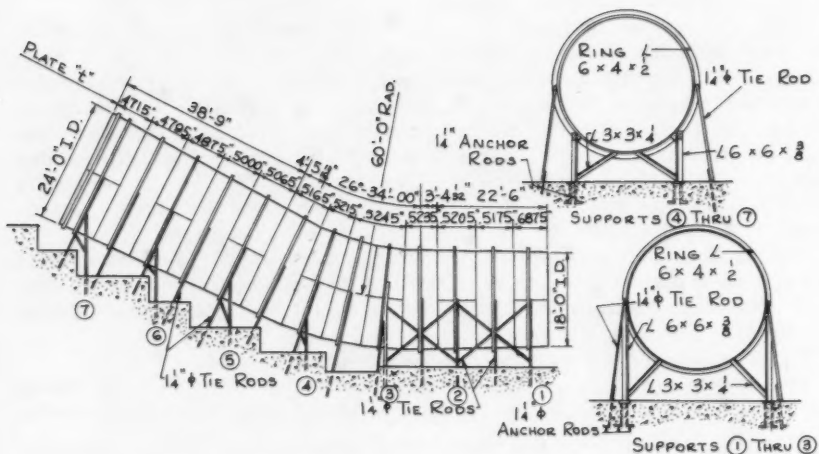
FIG. 14.—PUMP GENERATING STATION PENSTOCK

The plates are all butt-welded including attachment weld to the stay ring. Welding quality control and inspection was the same as that specified for the penstocks. Considerable bracing was required for erecting these scroll cases. Being of lighter construction the stay rings were more sensitive to erection and welding loads. Internal bracing remained inside the scroll cases until encasement was completed. Fig. 15 shows bracing in the scroll cases.

GENERAL COMMENTS

Shift work day and night for six days per week was required. Continuing erection at high speed during the very severe 1959 winter weather also was

FIG. 15.—INTERIOR BRACING SCROLL CASES PUMP GENERATING STATION

necessary to comply with schedules. On the Robert Moses Power Plant location, ice on the 48° rock slopes and repeated heavy snow falls resulted in dangerous working conditions and increased difficulties. Fig. 16 and Fig. 17 show winter conditions at Niagara. The wide spread concrete placement operations and rush completions for each of the five stages necessitated providing numerous unit crews with large amounts of equipment and welding machines dispersed up and down the 300 ft slopes and across the 1,100 ft forebay upper
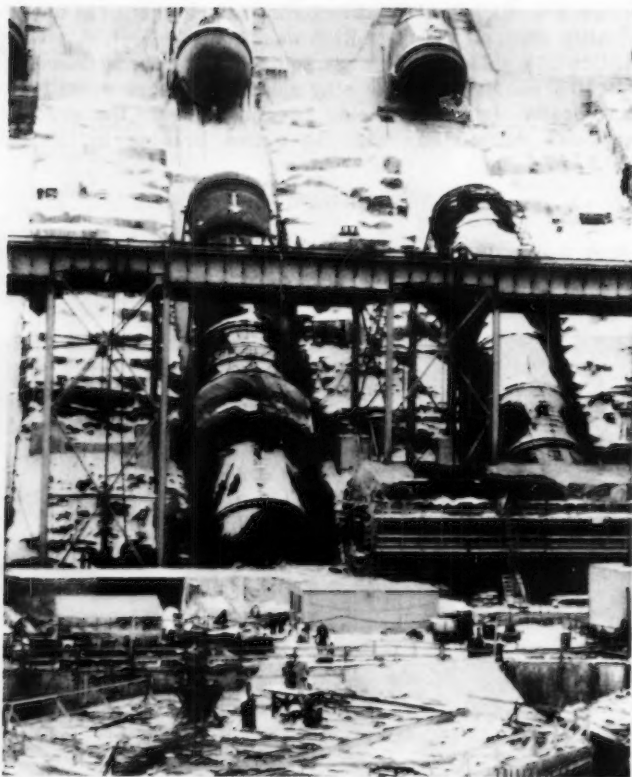
FIG. 16.—SNOW AND ICE CONDITIONS AT ROBERT MOSES GEN-
ERATING PLANT

FIG. 17.—WINTER CONDITIONS IN FABRICATING YARD

level and power house lower level. During peak progress periods penstock rings were installed in numerous isolated locations as fast as the concrete had set sufficiently to provide stable supports. Temporary supports were improvised so that penstock rings could be assembled before completion of the permanent anchors.

Two field fabrication yards at Lewiston were utilized to assemble and weld the 24 ft average diameter by 10 ft long rings. Fig. 18 shows an aerial view of one yard. Railroad and trucking clearances limited the shop fabrication to preparing half rings with half girders attached to every other plate ring. Fig. 19 shows material arriving at the assembly yard. Work at the field assembly yards and the shop at Greenville, Pennsylvania was started far in advance of the initial installation dates. Without the foresight which resulted in large inventories of completed rings at the site, it would have been impossible to meet the accelerations in penstock erection schedules resulting from the necessary changes in the general contractor's concrete placement programs. Each of the two subassembly yards and each of the two penstock and scroll case site construction operations were all independently staffed and equipped. The unprecedented high progress rates precluded the transfers of equipment that would have been characteristic of a normal penstock construction project. To avoid the penalties for delays, the equivalent of four separate construction projects had to be operated simultaneously and at maximum speeds. A total of 676 rings, 52 for each of the 13 penstocks for the Robert Moses Niagara Power Plant were assembled, welded, and 100% x-ray inspected at the large assembly yard. The hauling of component rings with weights as high as 31 tons each and outside diameters over the ring girders approximately 26 ft necessitated large crawler cranes and special truck trailers for transport and handling. Fig. 20 shows a truck trailer moving a ring to final location. A total of 132 rings, 11 for each of the 12 penstocks were assembled, welded, and spot x-ray inspected at the field fabrication yard for the penstocks and scroll cases for the pump generating plant.

Liquidated damages in case the penstock contractor was late in completing all five stages on each of the 13 penstocks including painting could have aggregated to $26,000 per day. This was based on $300 per day for construction delays and $100 per day for painting delays per each of the unit stages. This provided a challenge to the planning skills of engineers with long experience on penstock construction as it required completion at new record high rates of progress.

Corrosion protection consisted of one shop primer coat, field patch painting and two field coats of Federal Specification MIL-P-15145A zinc dust, zinc-oxide paint on inside surfaces combined with 1/16 in. corrosion allowance added to the design plate thicknesses, and thorough surface preparation by the phosphoric acid pickling process. A total thickness of the finished paint system averaging 3-3/4 mils to 4 mils and not less than 3-1/2 mils at any measured point was required. Special protection of weld edges against formation of new rust and scale expedited the welding operation. Little cleaning and brushing was required prior to welding. It was also found that the combination of phosphoric acid pickling and clear glyptal was more effective in protecting weld edges than was the combination of blasting and clear glyptal. The specified shop cleaning and priming was a factor that expedited field painting completion because the amount of field descaling and cleaning work was reduced to a minimum.

FIG. 18.—AERIAL VIEW OF ONE FABRICATING YARD

FIG. 19.—RAILROAD SHIPMENT OF FABRICATED RINGS
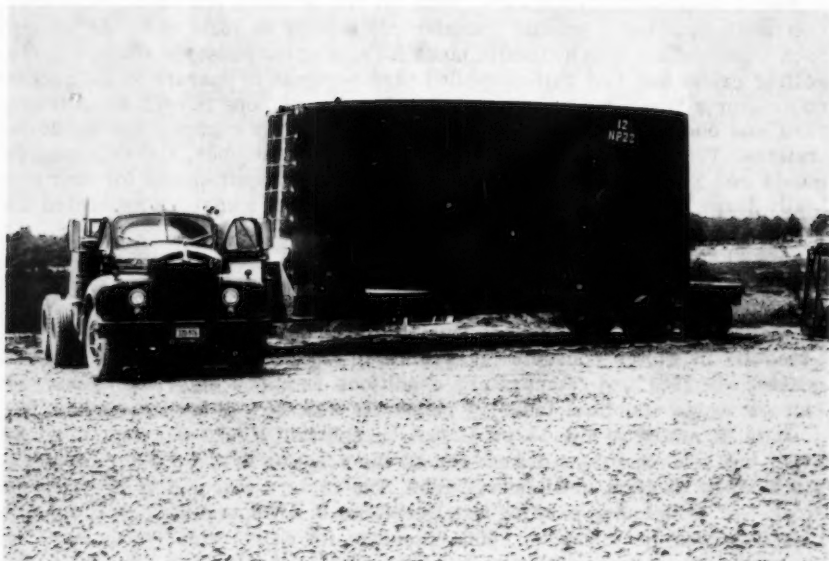
FIG. 20.—TRUCK TRAILER HAULING EQUIPMENT

FIG. 21.—GENERAL VIEW OF CONSTRUCTION AT ROBERT MOSES GENERATING
PLANT

Major construction equipment items for the penstock and scroll case erection work included a special transfer car moving on rails on the 48° sloping rock trenches and with hydraulic jacks for alining the penstock rings; a gantry rolling crane and two self-propelled cars designed to operate on the general contractor's trestles; three large crawler cranes, one in each subassembly yard and one at the forebay area; and several large capacity special design trailers. The total amounts of portable welding machines, air-arc-gougers, hoists and x-ray machines were equivalent to the requirements for four normally large penstock contracts, so as to allow the numerous separated unit crews to perform simultaneously on two shifts.

Actual completion time including the inside painting of penstocks No. 1 and No. 2 was about one month ahead of the specified time. In order to avoid any possibilities of delays on the concrete placement, most of the major work on the penstocks was finished well ahead of the scheduled completion dates. The reduction in winter concrete encasement in the last quarter of 1960 and first quarter of 1961 and temperature conditions required deferring making the closure welds and final tests on penstocks Nos. 8 to 13 until spring of 1961.

Date of award of the 13 penstocks on contract NP-22 was September 22, 1958, but site conditions did not permit the erection in place of the first penstock rings to start until June 3, 1959. The revised completion date for penstock No. 1 including painting was December 5, 1960, and the actual completion date was November 8, 1960. The date for delivery of first power from generator No. 1 was February 10, 1961. The final specified completion date for penstock No. 13 is December 1, 1961. Fig. 21 shows a general view of the construction site.

Contract NP-35 for 12 penstocks 18 ft to 24 ft diameter and 92 ft long each was awarded on February 24, 1959 and actual completion including painting was in the spring of 1961.

Completions on the scroll case erection subcontracts were all within the time allowances fixed by the turbine installation schedules.

The job is not yet finished (1961). But with the diligence and past performance of all contractors engaged on this monumental project it is almost certain the plants will be completed on schedule.

CONCLUSIONS

1. A large project such as is described herein requires meticulous planning and coordination to successfully meet rigid calendar schedules.
2. Construction procedures must be worked out and tried in advance of actual construction. A tight schedule requires fullscale performance at all times.
3. Job needs must be anticipated to avoid equipment and material shortage delays. Vigilance is the watchword.
4. The design engineer's intents must be thoroughly known and met so costly errors are not committed.

ACKNOWLEDGMENTS

York with Robert Moses, Chairman; William S. Chapin, General Manager and Chief Engineer; and William Latham, M. ASCE, Resident Engineer. Uhl, Hall & Rich were the consulting engineers. Penstock designs and specifications were supervised by George R. Rich, F. ASCE, A. E. Eckberg, F. ASCE, and C. Hebbel. On the penstock construction contracts at the site, J. P. Ottesen, F. ASCE, was Project Manager, J. P. O'Donnell, F. ASCE, Construction Manager and J. Carson assisted by C. Warrior supervised inspection.

The Chicago Bridge and Iron Company was the penstock contractor; J. Edgar Revelle, Contracting Engineer, Boston, Massachusetts; J. N. Pirok, F. ASCE, Design Engineer and Chief Structural Engineer; S. A. Johnson, Erection Manager; and H. A. Guerin, Project Engineer.

W. Cobb, assisted by F. Martell, was Superintendent on erection of 13 penstocks and scroll cases at the Robert Moses Niagara Power Plant.

W. Retzloff was Superintendent on erection of 12 penstocks and scroll cases at the Pump Generating Plant.

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FISH HANDLING FACILITIES FOR BAKER RIVER PROJECT

By Warner W. Wayne, Jr.,[1] M. ASCE

SYNOPSIS

The unique structures and equipment provided for collecting and transporting both upstream and downstream migrant salmon past the two relatively high concrete dams that form part of the Baker River Project are described. The artificial spawning beds, constructed to replace the natural salmon spawning areas near Baker Lake that were inundated when the new Upper Baker reservoir was filled are also described.

INTRODUCTION

The Baker River Project of Puget Sound Power & Light Company embodies two separate but contiguous medium head hydroelectric developments on the Baker River near the Town of Concrete, Wash. These two power developments, known as the Upper Baker River Development and the Lower Baker River Development provide a total of 198,350 kw of installed generating capacity for the Company power system. The location of the Baker River Project and other well-known points of interest in the State of Washington are shown in Fig. 1. A general plan of development for the Baker River Project, showing locations of the major facilities, is shown in Fig. 2.

A considerable portion of the Baker River watershed lies within the confines of the Mt. Baker National Forest, and during the spring and summer

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months the river is largely fed by glacial melt from the snow fields of Mt. Baker and Mt. Shuksan, two volcanic peaks rising 10,750 ft and 9,038 ft above sea level, respectively. The average yearly flow of the Baker River is 2,520 cfs. This river contains natural spawning areas for five species of migrating fish; four species of the Pacific salmon family and the steelhead trout. The salmon species consist of sockeye (Oncorhynchus Nerka), coho (0. Kisutch), pink (0. Gorbuscha), and Chinook (0. Tshawytsha). The sockeye and coho form the largest runs of migrating fish, with the pink salmon migrating only in the odd numbered years. The sockeye average 4 lb to 8 lb each and the coho,
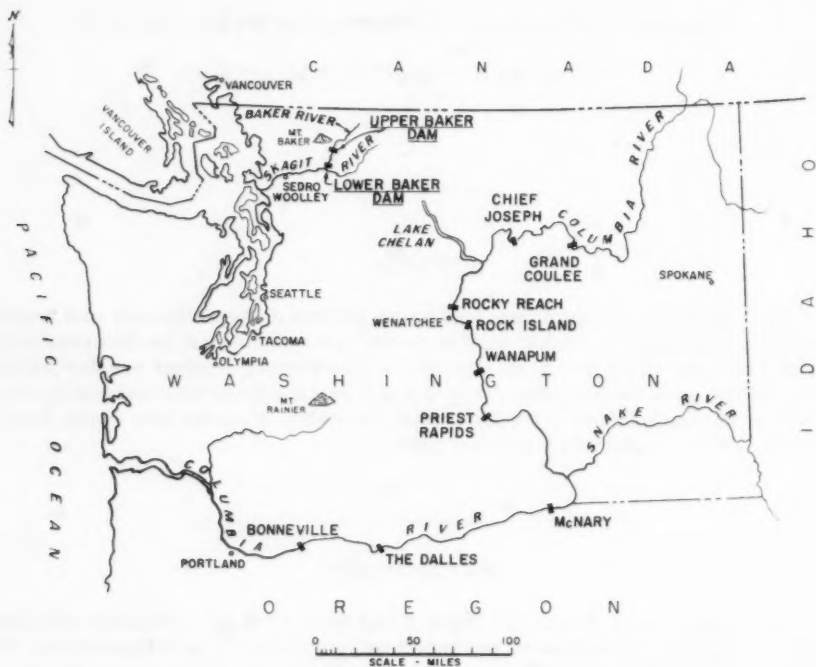
FIG. 1.—LOCATION MAP

often called silver salmon, average about 10 lb each, with both species being of considerable economic value to the commercial fishing industry. The sockeye are not usually caught by sportsmen, but the other species of salmon and the steelhead do contribute to sport fishing both in Puget Sound and the Skagit River.

The importance of the salmon fishing industry in the Pacific Northwest and the need to preserve and improve it has long been recognized by Federal State, municipal and private agencies. When plans were formulated in the early 1920's for the first power project on the Baker River, known as the

Lower Baker River Development, suitable facilities for collecting and transporting the migratory salmon past the dam were included. The magnitude of this handling problem can better be appreciated[2] when it is realized that (1) the average annual run of sockeye to the spawning grounds located in the Upper
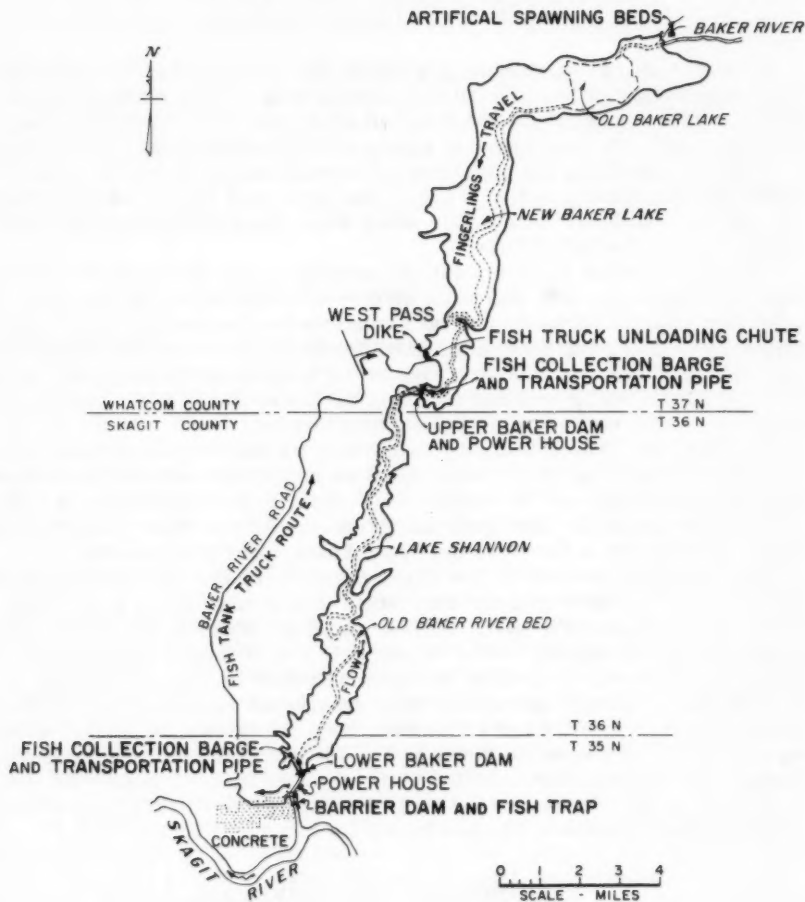
FIG. 2.—GENERAL PLAN OF DEVELOPMENT

Baker River near Baker Lake is about 3,000 and (2) the average annual run of coho to the spawning grounds located in the numerous tributaries which flow into Lake Shannon and Baker Lake is about 10,000. During unusually heavy fish runs, as many as 10,000 adult salmon have been handled during a single month and 3,000 in a single day.

2 "Annual Report," State of Washington Dept. of Fisheries, 1953.

To better comprehend the problems associated with the installation of new fish handling facilities for the Baker River Project, a brief description of both the Upper and Lower Baker River Developments follows.

BRIEF DESCRIPTION OF UPPER AND LOWER
BAKER RIVER DEVELOPMENTS

Construction of the original portion of the Lower Baker River Development was completed in 1927 and comprised a semigravity concrete arch dam with a maximum height of 285 ft and an intake structure for supplying water through a concrete lined tunnel to a powerhouse located about 1,200 ft downstream from the dam. The powerhouse contained two hydroelectric generating units rated 21,950 kva each, and provisions were made for the future installation of two additional units of the same size. The dam was then the seventh highest dam in the United States.[3]

The fish handling facilities for the original Lower Baker River Development consisted of a low diversion structure extending across the river just upstream from the powerhouse and a concrete fish ladder extending from the tailrace to a collecting pool on the east bank of the river. An 800 ft long highline cableway that was used to transport the trapped fish in small steel tanks between the collecting pool and the top of the Lower Baker dam where the fish were chuted into the reservoir was another feature.

In 1955, the Power Company authorized its engineers and constructors to undertake installation of a new 64,000 kw unit in an extension to the Lower Baker powerhouse and to design and construct a completely new hydroelectric development just upstream from the Lower Baker reservoir. The latter development is known as the Upper Baker River Development.

The principal features of the Upper Baker River Development include the following: a concrete gravity dam about 312 ft high with a crest length of 1,220 ft and containing about 600,000 cu yd of concrete; an integral intake structure for supplying water through $13\frac{1}{2}$ ft diameter penstocks to two hydraulic turbines; a spillway section controlled by three 25 ft wide by 30 ft high radial gates; and indoor type powerhouse containing two 47,200 kw turbine generating units and all necessary appurtenances; an outdoor switchyard; an earth dike about 60 ft high by 1,200 ft long; and facilities for handling downstream migrant fish, which will be examined in detail later. The reservoir formed by the Upper Baker dam is named New Baker Lake after the original lake inundated by the development.

RESEARCH AND TEST PROGRAMS

From the beginning of the power expansion program on the Baker River, it was realized that unusual methods and hitherto untried types of equipment might have to be utilized to collect and safely transport the upstream migrant salmon past the two high dams so they could reach their spawning areas as well as provide for safe passage of the young fingerlings past these same dams on their way downstream to the sea. Through mutual agreement, the

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[3] "Register of Dams in the United States," by T. W. Mermel, McGraw-Hill Book Company, Inc., New York, 1958.

Power Company provided the necessary funds to permit technicians of the State of Washington Department of Fisheries to conduct preliminary investigations and experiments and to conduct full scale field tests of special items of fish handling equipment which were being considered for the Baker River Project. Hydraulic research was carried on at the University of Washington laboratories, and field tests of experimental equipment and facilities were conducted at Lake Union, Mud Mountain Dam, Lower Baker Dam and Baker Lake.

With the information and experience gained through their comprehensive investigations and field tests, it was possible for the Department of Fisheries, working with the Power Company and their engineers, to establish an over-all plan covering the general types of fish handling facilities to be provided and to formulate basic design criteria. This program covered facilities for handling both upstream and downstream migrants, and to avoid possible confusion, they are described separately.

UPSTREAM MIGRANT FACILITIES

The term upstream migrants applies to adult salmon that are returning from their migration to the sea to their place of birth for the purpose of spawning. On entering fresh water they proceed upstream to the specific spawning area in which they were hatched, traveling at the rate of about 10 miles per day. On reaching the spawning area, the female normally deposits her eggs in a small depression which she prepares in the gravel bottom of a stream or lake, laying a total of from 2,000 to 5,000 eggs. These eggs are fertilized by the male, and on completion of the reproduction process both the male and female salmon die.

During the preliminary planning stage for the Baker River Project, a review was made of the functional efficiency and general serviceability of the existing Lower Baker fish handling facilities. Operating experience had proved that the capacity of these facilities was not adequate to handle the larger fish runs. Many fish died from overexhaustion in attempting to pass the diversion structure in the river before it was physically possible to collect and transport them by means of the overhead cableway. Due to deterioration of equipment and the extensive damage sustained by the diversion structure during recent floods, it was concluded that the existing facilities would require extensive and costly repairs to restore them to a satisfactory operating condition. For these and other reasons, the Power Company and the Department of Fisheries decided to abandon the old facilities and build a completely new barrier dam and fish trap about 1/2 mile farther downstream on the Baker River just above its confluence with the Skagit River, as shown in Fig. 2. Under the new plan, upstream migrants would be diverted by a barrier dam into an adjacent fish trap where they would be collected in a large steel hopper. They would then be transferred to special tank trucks for movement over roads past both the existing Lower Baker dam and the new Upper Baker dam and finally be released into New Baker Lake at the West Pass Dike. This procedure, although requiring a 14 mile haul over unpaved roads, eliminated the need for and expense of constructing additional facilities at the Upper Baker dam to handle these upstream migrants.

The functional design of the new fish trap was prepared by the State of Washington Department of Fisheries. It was essentially an adaptation of the Buckley type trap, which evolved over a period of 30 yr and has been used successfully on many Pacific Coast streams.

The engineering services for the new barrier dam and fish trap included preparation of final designs, specifications and detail drawings, and supervision of construction. A subcontract to construct these facilities was awarded to a general contracting firm. Work at the site began early in November, 1957 and was scheduled for completion by July 1, 1958, to make these new facilities available for handling the annual sockeye run that normally arrives in the Baker River during the first part of July. It was extremely urgent that the new fish trap be operating at that time, because the old fish trap was no longer usable and there would be no other way to handle these fish. The work was carried on vigorously, and despite flood difficulties experienced during the winter it was completed on schedule.

The general plan of the barrier dam and fish trap is shown in Fig. 3, and a photograph of the completed facilities is shown in Fig. 4. The barrier dam has a crest length of about 150 ft and has an unusual structural arrangement, being formed by removable precast concrete slabs resting on fixed concrete piers with the whole structure arranged to give a continuously unobstructed passageway for fish beneath the crest, as shown in Fig. 5. The dam crosses the river diagonally, and its upstream end is located adjacent to the fish trap entrance structure. Fish moving up the main river channel enter the passageway beneath the crest of the dam and are attracted to the fish trap entrance by the regulated discharge of attraction water over the entrance weirs.

One of the important design criteria established by the Department of Fisheries was that a minimum fall of 8 ft be provided at the barrier dam during the annual fish migrations that normally occur from July 1 to December 1. This height of fall was considered necessary to assure an effective barrier against the upstream passage of fish. Because the level of the pool created by the barrier dam affects tailwater levels at the Lower Baker powerhouse, it is desirable that the fixed crest of the dam be as low as feasible to provide maximum heads for power generation. With the selected crest fixed at 171 ft, it is possible to maintain the required differential head at the barrier dam except during heavy flood flows when backwater effects from the nearby Skagit River reduce the differential head. Hydraulic studies indicated that the installation of 2 ft high crest gates across the entire length of the dam would be sufficient to provide the necessary differential head under all expected flood conditions. It was decided to use two 75 ft long by 2 ft high radial gates for this purpose. When these gates are positioned below the fixed concrete crest there is no visual evidence of them from either shore because they are completely obscured by the water flowing over the dam. Fig. 5 shows the general arrangement of the radial gates. It will be noted that these gates differ from conventional installations in that the gate arms and trunnions and the center of curvature of the skin plates are all located upstream from the gate body, rather than downstream. With this arrangement, headwater pressure is applied to the concave side of the gate skin plate rather than the convex side, and the gate arms and trunnions are normally submerged in the headwater pool and thus are not visible. Most radial gate installations depend on the use of hoisting equipment supported from an overhead deck or bridge structures. However, the fixed hoists that operate the two radial gates are located on the concrete
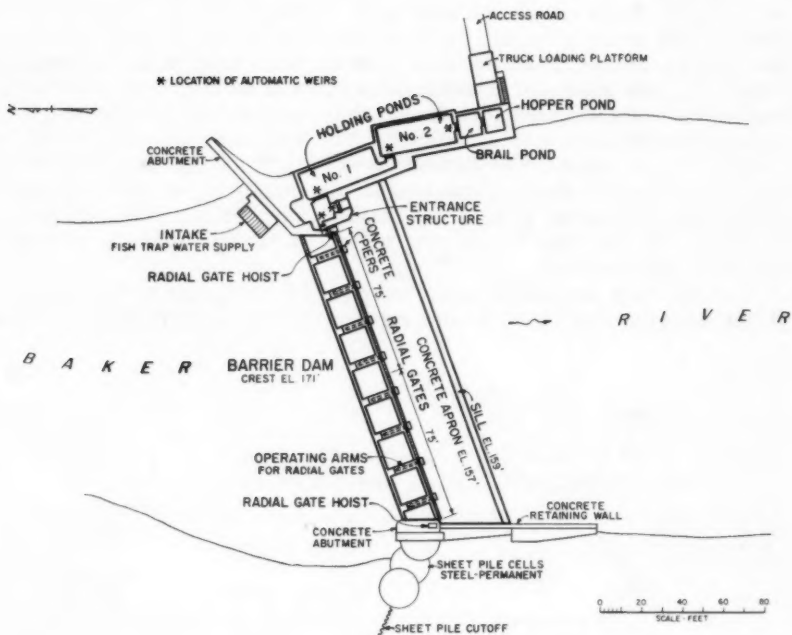
FIG. 3.—PLAN OF BARRIER DAM AND FISH TRAP

FIG. 4.—LOOKING UPSTREAM AT COMPLETED BARRIER DAM AND FISH TRAP

abutments of the dam, thereby effecting a considerable saving in cost as well as providing an overflow spillway which is completely free of piers or other obstructions above the crest level. As indicated in Fig. 5, each radial gate is operated by means of 4-wire rope cables. These are attached at their outer ends to the gate arms and extend inside a covered cable chase, provided in the precast concrete slabs that form the spillway crest, to a fixed hoist unit located on top of one of the concrete abutments. The hoist units are electrically operated and are provided with individual push button controls. Because the operating personnel are normally working in the vicinity of the fish trap, which is on the east side of the river, it was decided to provide an additional push button station for the west radial gate to permit remote operation of this gate from the east abutment.

The fish trap is located along the river bank adjacent to the easterly end of the barrier dam and is comprised of the following reinforced concrete
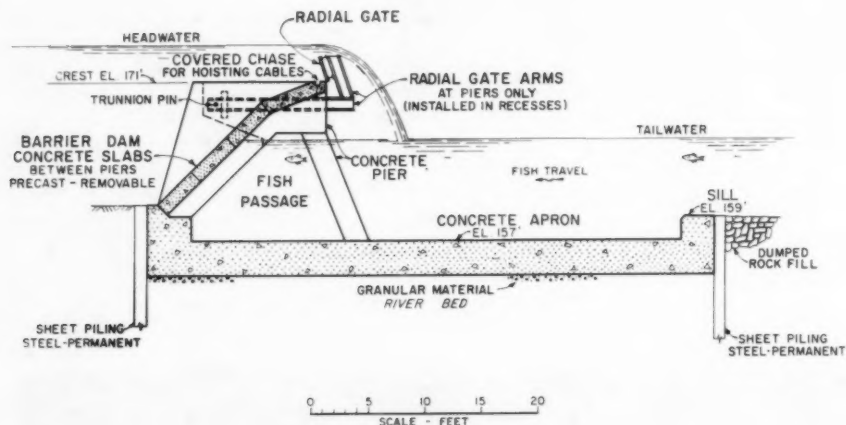
FIG. 5.—TYPICAL CROSS SECTION OF BARRIER DAM AND APRON

structures: a vestibule containing both main and auxiliary fishway entrances; two holding ponds, each 40 ft long by 15 ft wide; a brail pond 12 ft long by 12 ft wide; and a hopper pond 10 ft long by 12 ft wide. Above the hopper pond is a steel framed hoist structure about 44 ft high by 42 ft long which supports an 8 ton capacity overhead traveling hoist. This hoist is used to lift and transport a 1,000 gal steel hopper between the hopper pond, where the fish are collected, and a platform located at a higher level where the fish are transferred from the hopper to special tank trucks. Two such tank trucks are provided for transporting the fish between the fish trap and Baker Lake, where they are released. A longitudinal view of the fish trap, indicating the relative locations of the major items of fish handling equipment, is shown in Fig. 6.

To assure that fish will enter the fish trap and then progress upstream through holding ponds Nos. 1 and 2 into the brail pond and hopper pond, it is necessary to maintain an adequate flow of attraction water through the fish

FIG. 6.—LONGITUDINAL SECTION THRU FISH TRAP

NOTES:

1. EACH PAIR OF WEIRS IS OPERATED BY A SINGLE MOTOR OPERATED HOIST MOUNTED ON AN OVERHEAD STEEL PLATFORM AND AUTOMATICALLY CONTROLLED FROM A MASTER SELSYN. WEIRS HAVE STEEL FRAMES AND ARE CONCRETE FILLED

2. PICKET BARRIERS HAVE STEEL FRAMES SUPPORTING 1 x 4 WOOD SLATS SPACED TO GIVE 1" CLEAR OPENING BETWEEN SLATS. PICKET BARRIERS "R","S", "T" AND "U" ARE MOTOR OPERATED, "C" AND "D" ARE COUNTERWEIGHTED AND ARRANGED TO AUTOMATICALLY FOLLOW VERTICAL MOVEMENT OF FISH HOPPER

3. GATES "A" AND "B" HAVE STEEL FRAMES SUPPORTING A SOLID 1" TIMBER FACING. THE GATES ARE COUNTERWEIGHTED AND ARRANGED TO AUTOMATICALLY FOLLOW VERTICAL MOVEMENT OF BRAIL.

TYPICAL WEIR DETAIL

trap and to provide a suitable drop in water levels at the entrance to each pond. Attraction water is obtained from an intake structure located on the up-stream face of the east abutment of the dam, and the water moves by gravity flow through four separate underground conduits into the bottoms of the various ponds. Individual sluice gates are provided at the upstream ends of these four conduits for regulating the flows. The attraction water system is designed to supply a total of 80 cfs to the fish trap, apportioned as follows: 5 cfs to hopper pond, 5 cfs to brail pond, 10 cfs to holding pond No. 2, 40 cfs to holding pond No. 1, and 20 cfs to vestibule. To prevent excessive velocities at the points where the conduits discharge into the bottoms of the ponds, special diffusion chambers are provided to spread the flow uniformly over a large area and re-duce the average exit velocity to about 1/4 cfs. In this way, it is possible to minimize the likelihood of fish congregating at these points with resultant delays in their upstream movement.

The required drop in water levels at the entrance to each pond is obtained by means of electric hoist operated weirs located in the water passages. These weirs are essentially small slide gates constructed of steel and filled with concrete. A total of fi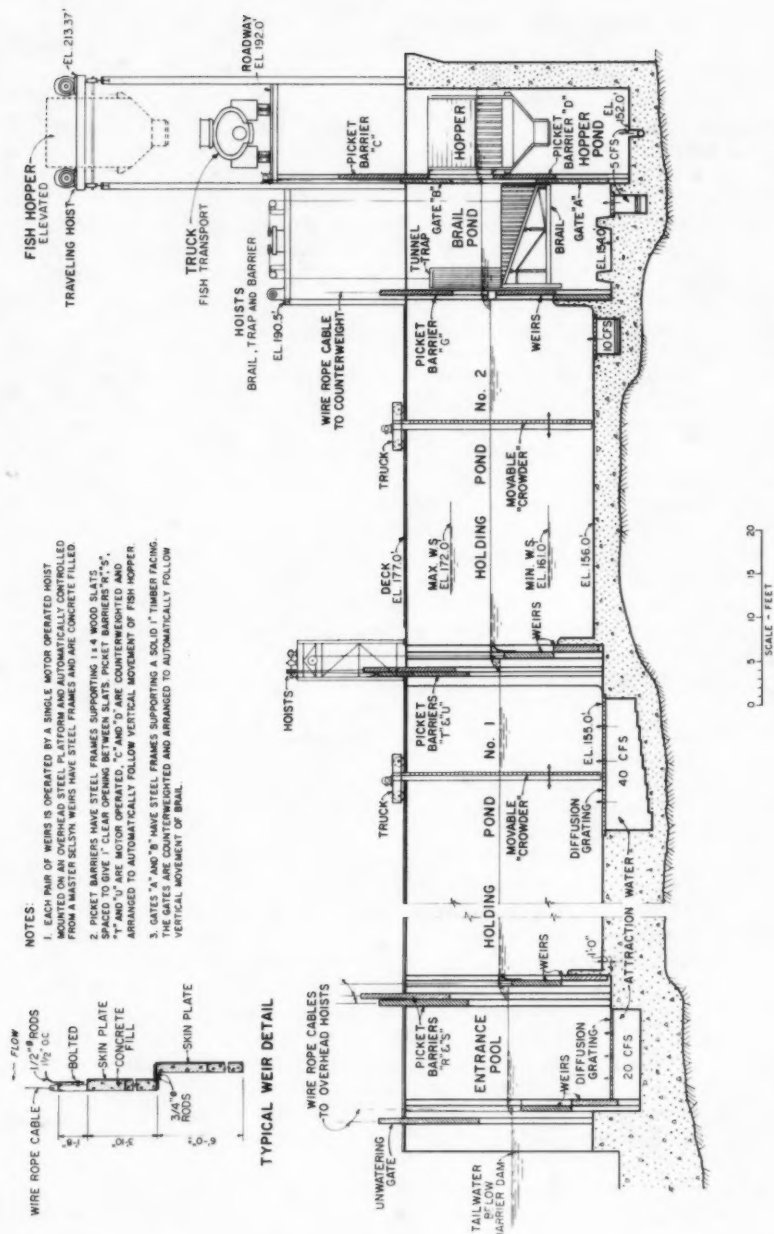ve such weirs are provided. They are automatically con-trolled to follow the rise and fall of tailwater levels in the Baker River and to provide a constant differential head at each weir regardless of the river stage. Because tailwater levels are apt to fluctuate through a considerable range each day due to variable power plant releases, the automatic control system elimi-nates the need for, and expense of, a permanent attendant to adjust these weirs manually to suit different river conditions.

The automatic control system has evoked much interest from engineers and technicians responsible for the design and operation of fish handling facili-ties in the Pacific Northwest. It represents the most elaborate application to date (1961) of the Selsyn differential system for controlling equipment of this nature. The basic Selsyn differential system for the automatic weirs consists of a Selsyn transmitter, a differential Selsyn, and a Selsyn receiver which are electrically interconnected and operate in a manner similar to that of a simple Selsyn system. The voltage distribution in the primary winding is the same as that in the secondary winding of the exciter Selsyn. If any one of the three Selsyns is fixed in position and a second one displaced through a certain angle, the third, being free to rotate, will turn through the same angle. For the instal-lation being described, there is one master Selsyn transmitter that is activated by a tailwater float, five differential Selsyns and five Selsyn receivers complete with control panels and electronic relays. Each Selsyn receiver is mechanically connected to the weir that it controls, and its rotation is directly proportional to the vertical travel of the weir. When the master Selsyn is rotated out of cor-respondence with the receivers due to movement of the tailwater float, each receiver sends out a directional signal to its control panel. The signal is amplified and operates a telephone type relay which, in turn, causes the hoist driving motor to operate until the movement of the weir is sufficient to rotate the Selsyn receiver into correspondence with the master Selsyn. At this time the signal will disappear, the relay will drop out, and the positioning motor will come to a stop. The differential Selsyns provide a simple means for electrically adjusting the relative height of each weir with respect to tailwater level. Once these adjustments are made, the differential Selsyns remain fixed in position and the operating signals affect only the Selsyn receivers. Hydraulic studies indicated that the desired flow characteristics in the fish trap would be

achieved by setting the crests of the main and auxiliary entrance weirs 4 ft below tailwater level and by setting the crests of the weirs at the entrances to holding ponds Nos. 1 and 2 and the brail pond at about 0.0 ft, 1.0 ft and 2.0 ft above tailwater level, respectively. These settings have been used and found to be satisfactory.

Because previous experience at other fish traps had shown that some fish could be expected to linger in the large holding ponds and not proceed upstream to the brail pond where they could be trapped, it was considered desirable to install "crowders" (followers) in the two holding ponds which could be used to force any loitering fish to move upstream into the next pond. Each of these movable "crowders" is about 15 ft wide by 22 ft high and is fabricated from standard 1 1/4 in. by 3/16 in. steel grating welded to a steel supporting frame. The main frame is suspended from a motor driven carriage which rides on rails located on top of the side walls of the holding pond. Holding pond No. 1 and the carriage and upper section of a "crowder" are shown in Fig. 7.

Normally, each "crowder" is stored at the downstream end of its respective holding pond and beyond the point where fish enter the pond, thus causing no interference with the normal passage of fish through the pond. When it is necessary to utilize a "crowder," the entrance to the holding pond is first closed by lowering a picketed barrier across the opening. This prevents fish that have entered the holding pond from swimming back out as the "crowder" starts to move. By operating the "crowders" in holding ponds Nos. 1 and 2 in successive order, it is possible to clear both ponds of any lingering fish and to force them finally into the brail pond.

The brail is a special item of equipment that performs a function similar to that of the "crowder," except that its travel is vertical instead of horizontal. Its bottom and sides are constructed of wood grating to permit free drainage of water when it is lifted, thereby reducing the hoist load. When a sufficient number of fish have entered the brail pond, the entrance is blocked by a picketed barrier and the brail is raised vertically by a fixed hoist mounted directly overhead on a steel platform. The bottom of the brail is inclined towards the hopper pond and, as it is raised out of water, the fish tend to slide and swim in an upstream direction directly into a fish collecting hopper. Fig. 8 shows a salmon lying on the wood grating at the bottom of the brail and adjacent to the entrance to the fish hopper.

The fish collecting hopper is basically a 1,000 gal steel bin with open wood grating sides extending about 3 ft above the top of the steel plates, followed by a 5 1/2 ft height of solid timber planking. A small rectangular opening is provided in the wood grating on the downstream side of the hopper and when the adjacent brail is raised, the fish swim from the brail through this small opening into the top of the hopper. When the maximum of about 100 adult salmon have been collected in the hopper, the entrance is closed by a small wooden slide gate and the hopper is raised out of the water by an 8 ton capacity overhead traveling hoist. The hoist is electrically operated and controlled by push button stations. Limit switches are provided to automatically control the horizontal and vertical limits of travel of the hopper and make it possible to position the hopper accurately over the tank truck each time. Thus, the discharge column on the bottom of the hopper will properly seat itself within a special watertight mating collar built into the top of the 1,000 gal tank on the fish transport truck. While engaged, the tank is first filled with water pumped from the river before opening the discharge valve in the bottom of the hopper, this prevents a rush of
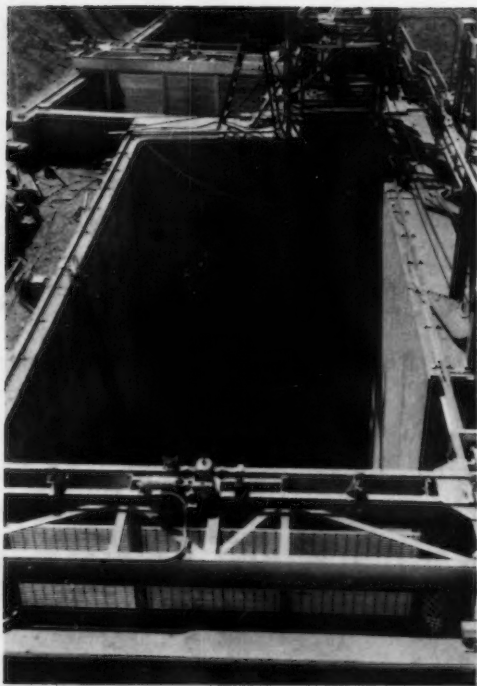
FIG. 7.—VIEW OF HOLDING POND NO. 1 SHOWING PIC-
KET BARRIERS AT ENTRANCE TO HOLDING
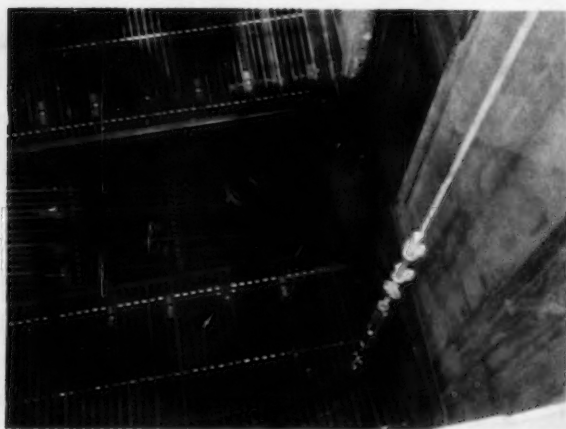POND NO. 2 AND TOP OF "CROWDER" IN
FOREGROUND

FIG. 8.—VIEW OF BRAIL SHOWING A SALMON ON BOTTOM GRATING
ADJACENT TO ENTRANCE TO FISH HOPPER

water from the hopper into the tank which could result in injury to the fish. After the hopper valve is opened, the next step is to open a screened drain valve located in the bottom of the tank. This makes it possible to transfer slowly the water and fish from the hopper to the tank. The drain valve is closed as soon as the hopper is completely emptied. One complete loading cycle, including raising the hopper, transporting it laterally, lowering it to the truck level, filling the tank truck, and transferring the fish from the hopper to the tank truck, takes approximately 8 min.

The fish transport trucks are modified versions of similar tank trucks provided for hauling fish at other power projects in the Pacific Northwest and are considered part of the Buckley type trapping system. A photograph of a typical truck of this type is shown in Fig. 9.

Both trucks furnished for the Baker River Project consist of a G.M.C. Model W504-A truck chassis on which a specially constructed 1,000 gal capacity steel tank is mounted. To facilitate loading of fish, a 40 in. diameter mechanically operated hatch cover is provided at the top of the tank. The fish are discharged through a quick-opening rear door arranged to lock automatically in the open position. A 4 in. diameter drain line with a manually operated gate valve is provided for emptying the tank, and the inlet to the drain line is screened to prevent fish from entering.

Two complete water circulation systems are provided for each tank, so that if one should fail during the long road trip, the other would be readily available and the possible loss of a complete load of fish could be prevented. Each water circulation system consists of an independent gasoline driven pumping unit capable of circulating all the water in the tank in 8 min to 10 min. A 2,000 lb capacity ice compartment is provided at the front end of the tank for use during extremely hot weather when it may be necessary to cool the water in the tank. Manually operated valves are installed in the pump suction lines. This permits drawing water from either the fish tank or ice compartment or mixing in any desired proportions according to the temperature required. Water temperatures in the fish tank and pump outlet are indicated by gages installed on the dashboard of the truck. It is essential that the dissolved oxygen content of the water be maintained at a sufficiently high level to prevent loss of fish in transit. For this reason, a Venturi type air injector is installed between the circulating water pump and the intake for aerating the water. Air is furnished to the injector by copper tubing which extends above the maximum water level in the tank, and as water passes through the injector the suction produced in the tubing causes air to be mixed with the water. The incoming mixture of water and air from the circulating water system enters the bottom of the tank through a distributor made up of perforated pipes that limit the entrance velocities to about 1 fps. By diffusing the water in this manner, the fish are not likely to be attracted to, or congregate about, the outlets for the two circulating water systems. The piping arrangement also provides for emergency filling of the tank by means of a flexible hose. The hose can be attached to the pump suction at one end while the other end is lowered into a pond, stream, or other source of water.

ARTIFICIAL SPAWNING BEDS

The Upper Baker dam, completed in July, 1959, created a reservoir 9 miles long with a normal full pool elevation of 724 ft. This project raised the level of

FIG. 9.—SIDE VIEW OF FISH TRANSPORT TRUCK SIMILAR TO TYPE USED
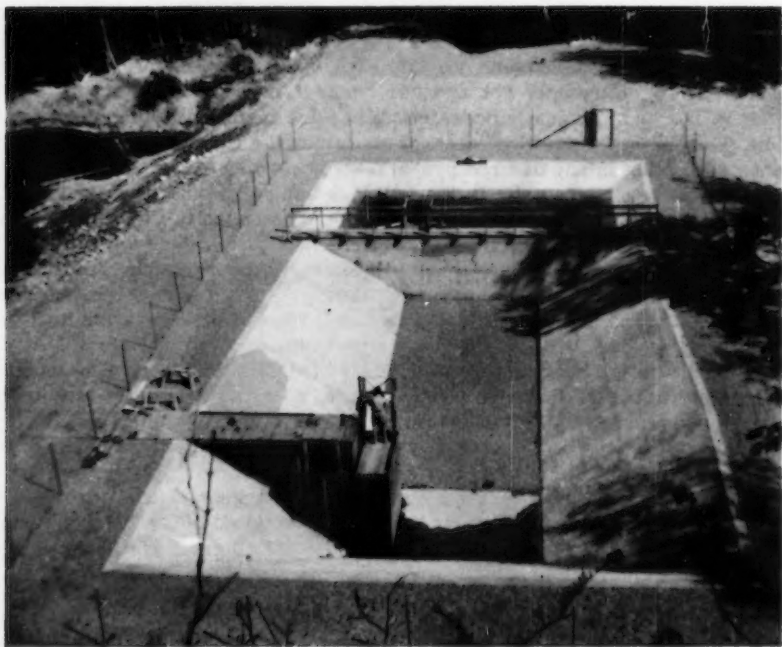FOR BAKER RIVER PROJECT

FIG. 10.—VIEW OF ARTIFICIAL SPAWNING BED NO. 1 WHILE UNWATERED

the original Baker Lake by 60 ft and rendered useless the natural spawning areas for sockeye salmon that had been located along the north shore of the lake. Recognizing the replacement of these spawning areas would be necessary to assure continued propagation of sockeye, the Department of Fisheries had decided earlier to investigate the possible use of artificial spawning beds for this purpose. A suitable site for an experimental spawning bed was selected adjacent to Channel Creek, a small tributary which flows into the Baker River just north of Baker Lake. Construction of a 50 ft long by 20 ft wide concrete lined spawning pool with sloping sides was started during the fall of 1956 and completed in the spring of 1957. A timber diffusion chamber was constructed on the concrete bottom of the pool, and three graded gravel filter blankets having a total depth of 42 in. were laid above this chamber. Water was supplied to the upper end of the diffusion chamber and percolated slowly upwards through the gravel bed. This simulated flow conditions normally found at natural fish spawning grounds. Other supplementary facilities were constructed to fulfill operating requirements, including a small diversionary structure built across a branch of Channel Creek, a 12 in. diameter water supply pipe about 250 ft long, a reservoir pool with regulating weir for controlling flows into the spawning pool, and outlet works for controlling the releases of water back into Channel Creek. Fig. 10 shows the completed experimental artificial spawning bed.

The results of the first year's operation of the artificial spawning bed were most satisfying and indicated that of the estimated eggs spawned, 43% resulted in survival of fry. This survival rate is about the same as those of natural spawning beds as determined from studies by the International Pacific Sockeye Salmon Commission in Canada. The first fry appeared on the artificial spawning beach late in January, 1958, which corresponded closely to the arrival date of fry on the natural spawning grounds in the area that year. Although this emergence of fry was about one month earlier than usual, it was attributed to above normal winter temperatures.

The success of artificial spawning bed No. 1 led to a joint decision by Power Company and the Department of Fisheries to extend the artificial spawning area to include a larger bed having a pool 150 ft long by 100 ft wide with sloping earth sides. Plans and details of the artificial spawning beds are shown in Fig. 11.

Construction of spawning bed No. 2 was started during late 1958 and completed in July, 1959. Spawning bed No. 2 was excavated in the natural gravels present at the site and the excavated material used to construct the earth embankments that form the four sides. Because the foundation material was found to be fairly pervious, it was concluded that the available supply of water that could be diverted into the bed would not be sufficient to maintain the required pool depth, unless some type of impervious bottom seal were used to prevent excessive seepage losses.

The engineers reviewed several methods of constructing such a seal and decided to utilize sheets of polyethylene film for this purpose. A 6 in. layer of sand was placed over the gravel bottom and sides of the spawning pool, and the polyethylene membrane was laid on this sand and then covered with another 6 in. layer of sand. These sand blankets minimize possible damage to the membrane by sharp stones or other objects. The actural spawning bed was constructed of special graded gravels placed above the membrane on the bottom
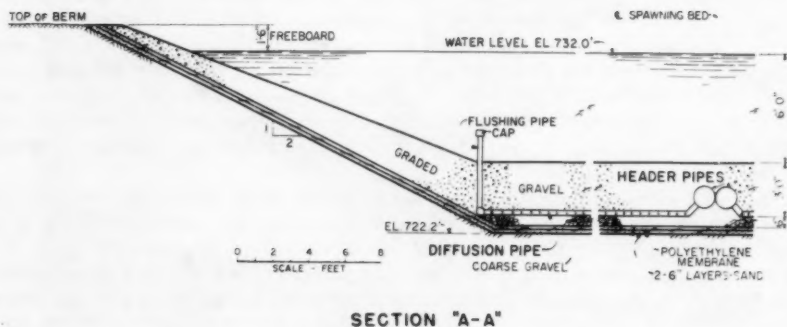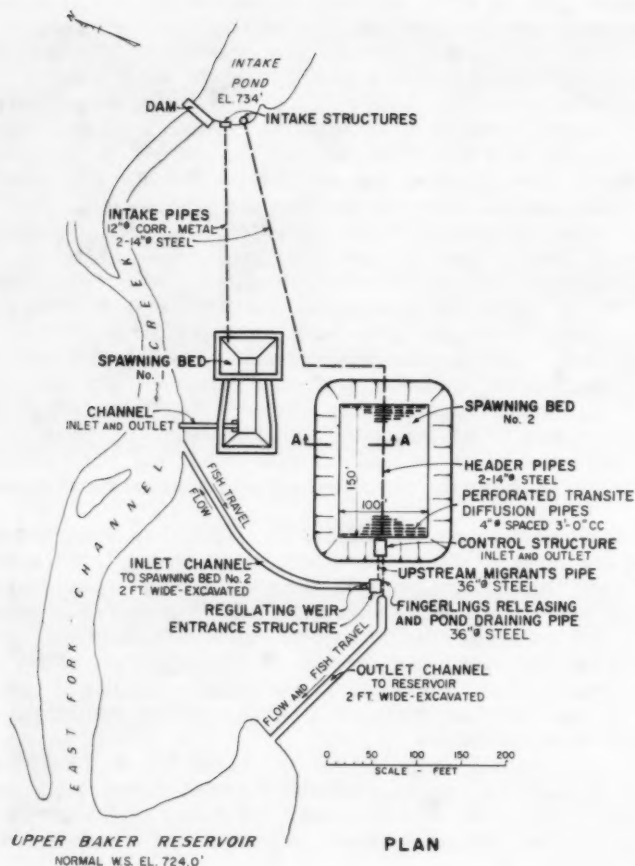
FIG. 11.—PLAN AND DETAILS OF ARTIFICIAL SPAWNING BEDS

and sloping sides of the pool. Water is fed into the bottom of the spawning bed through diffusion piping and allowed to percolate slowly upward through the gravels to simulate flow conditions at natural spawning sites. The diffusion piping system is quite extensive and consists of two 14 in. diameter steel pipe headers extending lengthwise down the pool and 4 in. diameter transite laterals connected to the header on both sides at 3 ft intervals. The pipe laterals are perforated with 11/64 in. diameter holes at 8 in. intervals.

A timber control structure was built at the downstream end of spawning bed No. 2 for regulating discharges from the pool and to serve as an entrance for adult salmon moving up Channel Creek from the adjacent New Baker Lake to spawn. A 2 ft wide ditch with sloping sides was excavated between Channel Creek and the control structure to provide the necessary waterway for these salmon. Another 2 ft wide drainage ditch was constructed at a lower level between the control structure and the nearby shore of the lake for use in un-watering the spawning bed.

The entire life cycle of the sockeye salmon of the Baker River system, including its stay in fresh water as a fingerling, migration to the sea, and eventual return to its place of origin for the purpose of spawning normally covers a period of 4 yr. Therefore, to assure a complete transition of the sockeye runs from the natural spawning areas to the two new artificial beds, it is necessary to transport by truck and release directly into these artificial beds all sockeye salmon collected at the barrier dam and fish trap during its first 4 yr of operation. On this basis, it is expected that by 1963 and thereafter, all adult sockeye moving upstream to spawn on the Baker River will return by natural instinct to the artificial spawning beds where they were hatched.

DOWNSTREAM MIGRANT FACILITIES

Salmon eggs normally hatch within 40 days to 60 days depending on the water temperature. The newly hatched fry begin life in the gravels where they are born and later, as "fingerlings," move into the open streams to live in fresh water for varying periods of time. The coho, pinks, and fall Chinook migrate to the sea in their first year, but the majority of Baker River sockeye do not leave for the sea until their second year. The downstream migrant facilities described in this paper were considered necessary to assure the safe passage of these fingerlings over the two relatively high dams that lie in their path.

During construction of the original Lower Baker River Development, no special facilities were provided for handling downstream migrants at the main dam. Most of the fingerlings were discharged over the spillway during the spring floods, but others entered the power tunnel and were discharged through the hydraulic turbines. On occasion, dead or injured fish had been noted below the dam and in the powerhouse tailrace, and it is reasonable to assume that a decline in the sockeye run which occurred after the Lower Baker dam was constructed was due in part to the fingerlings being injured or killed while passing over the dam or through the turbines. A field study was made during the period 1950 to 1952, inclusive, by the State of Washington Department of Fisheries in cooperation with the International Pacific Salmon Fisheries Commission, to determine the pattern of migration from the Lower Baker reservoir, the effect of the spillway and Francis type turbines on survival, and the

most probable causes of mortality.[4] The results of this study indicated that 64% of the sockeye and 54% of the coho passing over the spillway were killed. For the very small proportion of fingerlings that passed through the tunnel and turbines, the mortality rate was about 34% for sockeye and 28% for coho.

Subsequent tests conducted at Glines Dam on the Elwha River in Washington during 1952 and 1953,[5] and experiments in dropping fingerlings up to 6 in. long from a helicopter into a body of water, indicated that salmon fingerlings could, under free fall conditions, be safely passed over any height dam provided the tailwater pool were sufficiently deep. A more definitive type of test was conducted in October, 1955 at the Lower Baker dam using an experimental timber ski jump type spillway chute for passing the salmon fingerlings between fore-bay and tailwater under conditions approximating free fall. A total of 240,000 fingerlings were passed over the ski jump spillway during the test, and a survival rate of 85% was obtained compared to about 40% for fish passing over the conventional spillway section.[6] Fig. 12 shows the ski jump spillway in operation.

Although very favorable results were obtained with the ski jump spillway, the release of 500 cfs-600 cfs of water with equivalent loss of potential power combined with limitations imposed by reservoir operating conditions made this method of handling fingerlings undesirable. Meanwhile, the Department of Fisheries was studying the possibility of using fish collection barges at both the Upper and Lower Baker reservoirs for handling downstream migrants. As part of this study, a full size experimental barge was built by the Department of Fisheries and tested by them in the Mud Mountain reservoir to determine whether fish would be collected, to find suitable entrance velocities, to check the effectiveness of louvers in guiding fish into selected paths, and to establish basic design criteria for a prototype barge. On successful completion of these tests, it was mutually agreed to adopt this new method of handling fish and functional plans were prepared by fishery technicians for a 68 ft long by 36 ft wide fish collection barge to be installed at the Lower Baker reservoir as a permanent part of the downstream migrant facilities. Final designs and detail drawings of the fish collection barge were prepared by the engineers. Field erection of the barge was also performed by the engineers.

The Lower Baker fish collection barge was initially placed in service during early April, 1958 to handle the annual downstream migration of fingerlings. Fig. 13 shows the barge in operating position in the reservoir. Following the first season of operation, the barge flotation system was modified to improve barge stability and provide additional buoyancy. Before proceeding with preparation of plans for another fish collection barge to be used at Baker Lake, it was mutually agreed to wait until another season's operation had been completed and the results evaluated.

From the experience gained during these first two seasons of fish collecting activities at Lower Baker, it was concluded that a greater quantity

4 "An Investigation of the Effect of Baker Dam on Downstream Migrant Salmon," by J. A. R. Hamilton and F. J. Andrew, Internatl. Pacific Salmon Fisheries Comm., Bulletin VI, 1954.

5 "Investigation of Mortalities to Downstream Migrant Salmon at Two Dams on the Elwha River," by D. E. Schoeneman and C. O. Junge, Jr., State of Washington Dept. of Fisheries, Research Bulletin No. 3, April, 1954.

6 "Report on the Test of a Ski Jump Spillway, Baker Dam," by A. F. Regenthal, Fisheries Biologist, State of Washington Dept. of Fisheries, 1955.
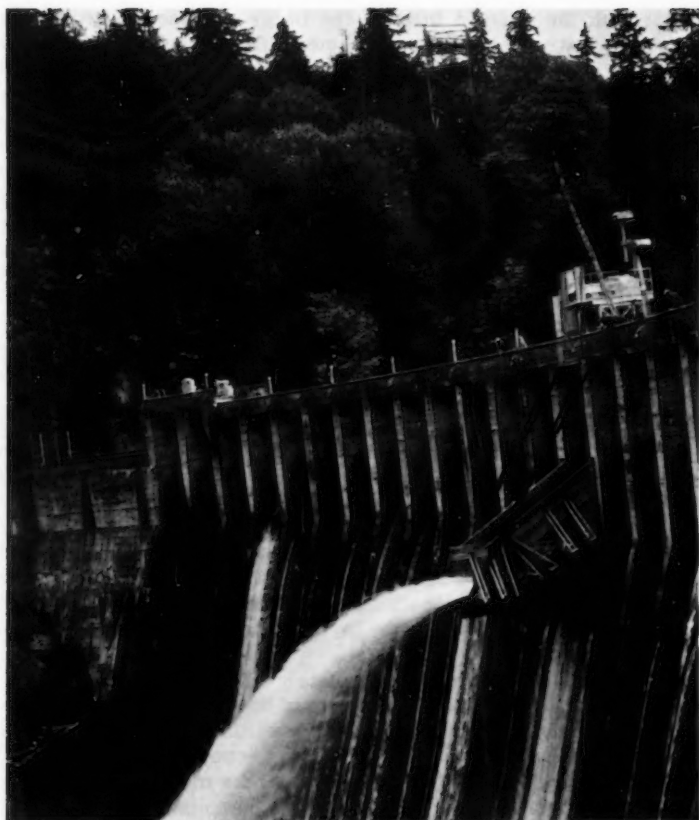
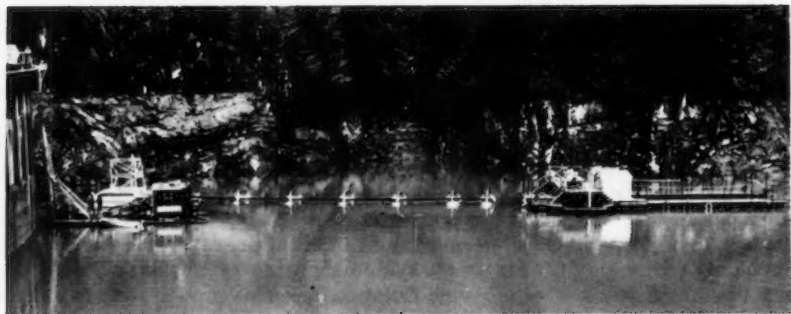FIG. 12.—LOWER BAKER SKI-JUMP SPILLWAY IN OPERATION

FIG. 13.—LOWER BAKER FISH COLLECTION BARGE IN OPERATING POSITION
IN LAKE SHANNON

of water should be pumped through the barge entrance in order to increase the attraction water velocities. This meant that larger size attraction water pumps would have to be installed in the barges. For this reason, and because of certain inadequacies found in the original barge flotation system, it was considered desirable that a completely new design be developed for the Upper Baker fish collection barge. Preliminary functional drawings of the new barge were prepared by the Department of Fisheries, with detailed plans and specifications being prepared by the engineers. The final design was completed in late 1959 and a contract placed for the construction of a barge approximately 36 ft wide by 70 ft long. The same firm had built the Lower Baker fish collection barge. Shop fabrication and field erection were expedited, and the barge was launched in New Baker Lake early in April, 1960, just before the start of the annual downstream migration of salmon.

The general arrangement and details of the Upper Baker fish collection barge are shown in Fig. 14 and 15, respectively. The main barge structure is comprised of light angle trusses that were shop welded into box type assemblies and arranged for bolting together in the field. A total of 28 steel flotation tanks are installed inside the steel truss assemblies to provide the necessary buoyancy for supporting the barge loads under various operating conditions. Fourteen of these tanks are 60 in. inside diameter by 12 ft long and fourteen are 38 in. inside diameter by 5 ft long, the larger tanks being installed in a vertical position and the smaller tanks in a horizontal position. Additional buoyancy is provided by a large number of Styrofoam planks attached to the barge framing at selected locations. Cathodic protection is provided for the steel framing and steel tanks by a well distributed system of magnesium ribbon anodes.

When the flotation tanks are completely filled with air, the barge draft is slightly less than 4 ft so that the bottom of the fishway channel and all barge operating equipment are above water level and accessible for inspection, maintenance, or repairs. In order to attain the normal fishway position, it is necessary to lower the barge in the water until the main operating deck is only about 1 ft above the reservoir level. This is accomplished by opening the drain valves in the bottom of the flotation tanks and admitting water slowly into these tanks until the desired depth of submergence is reached. When it is desired to reverse the process, or raise the barge in the water, low pressure air is admitted into the top of the tanks and it forces the water out through the bottom drains. As soon as the water has been discharged from the tanks, the drain valves are closed.

Low pressure air for the flotation tanks is supplied by a rotary type positive displacement blower having a rated capacity of about 100 cfm of free air and a discharge pressure of 10 psi gage. The blower motor is automatically controlled by a pressure switch that is arranged to start the motor when the pressure in the air line drops to about 5 psi gage and to stop the motor when the pressure reaches 10 psi gage. Pressure relief valves are provided in the air supply piping and are set to open in the event air pressures should exceed about 12 psi gage, which is the maximum internal pressure the flotation tanks can safely withstand. The air supply system includes two separate pipe headers with individual feeder lines to each flotation tank. One pipe header is arranged to supply air to the 14 large vertical tanks, and the other header supplies air to the 14 small horizontal tanks. Separate shutoff valves and exhaust valves are provided for each header to give greater flexibility of operation.
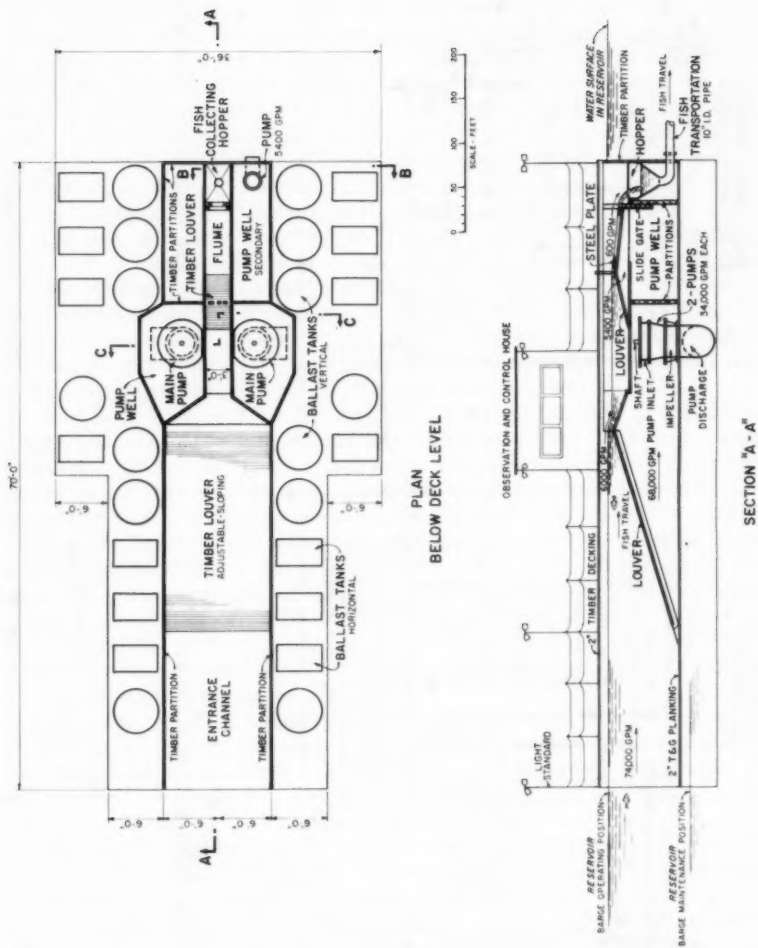
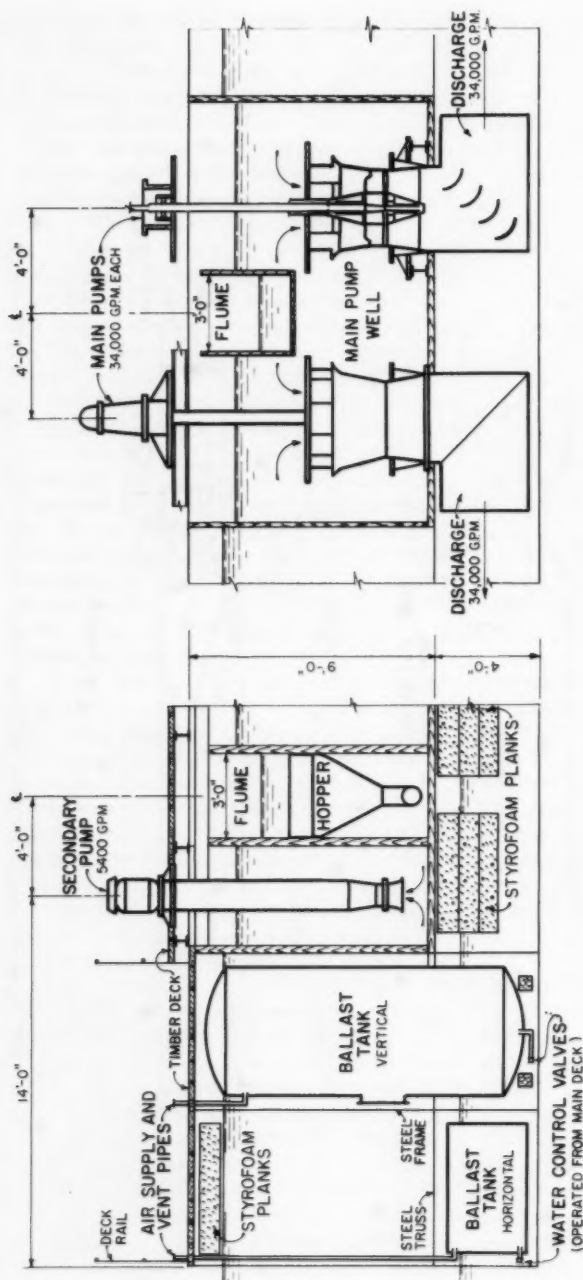FIG. 14.—GENERAL ARRANGEMENT OF UPPER BAKER FISH COLLECTION BARGE

FIG. 15.—DETAILS OF UPPER BAKER FISH COLLECTION BARGE

Two large turbine pumps rated 34,000 gpm (75 cfs) each and a secondary pump with a capacity of 5,400 gpm (12 cfs) are used to create a substantial flow of reservoir water through the barge entrance. It is believed that the fingerlings are attracted to the barge by this induced current. It is this premise that led to the original design concept. The pumps are arranged to discharge in opposite directions, so as to balance the hydraulic thrusts under normal operating conditions, and the water is returned directly back into the reservoir. The total pumping head is only about 5 ft, and it consists mainly of hydraulic losses in the pump and the discharge velocity head. Installation details for the two main pumps are shown in Fig. 15. These pumps are constructed of Cor-Ten steel throughout, in order to reduce the weight as well as to provide a corrosion resistant material. Pump bearings are of cutless rubber and are lubricated by the water being pumped. The pump drivers are mounted overhead on a steel bridge structure and consist of right angle gear drives directly connected to 75 hp totally enclosed horizontal shaft motors through flexible couplings. The pumps are normally operated on a continuous basis during the period of fish migration. In the event one large pump should be out of service for any reason, the remaining pump has sufficient capacity to permit fish collecting activities to continue. However, the barge must be raised about 2 ft in the water to reduce the effective area of waterway and thereby maintain sufficiently high entrance velocities to attract fish. The fish handling equipment has been made adjustable to accommodate this 2 ft range in barge operating levels.

The main waterway, or fish passage, is about 12 ft wide by 9 ft high by 42 ft long, and the sides and bottom are lined with 2 in. thick tongue and groove planking. A sloping timber louver is installed across the inner end of this waterway to prevent the fingerlings from entering the pump well for the two large pumps. At the top of the louver is a flared steel chute that leads to a 3 ft wide steel flume. About 6,000 gpm of attraction water is allowed to pass over the top of the louver into the chute and flume, and it is in this flowing water that the initial trapping of fish occurs. Before reaching the end of the flume, about 5,400 gpm is diverted into a secondary pump well located directly below the flume. From there it is discharged by the 5,400 gpm pump back into the reservoir. A sloping timber louver, or screen, is installed over the entrance to the secondary pump well to prevent the loss of fingerlings into this well. The remaining 600 gpm of attraction water contains all the fingerlings that have been trapped, and this water is finally discharged into a steel hopper located at the barge stern. The water and fish are then discharged through the bottom of the hopper into a 10 in. diameter flexible pipe which is used to transport the fingerlings between the barge and the Upper Baker dam.

The barge superstructure includes timber deck areas, a pump bridge, and a combination observation and control house. The control house contains the rotary blower and various items of electrical equipment, including the incoming power supply leads, step-down transformer, terminal box, and motor control panels. Large sliding glass windows are provided in the control house wall which fronts on the main waterway to permit barge operating personnel to observe the passage of fish into the entrance to the flume. Electrical space heaters are provided inside the house for use during cold weather. Floodlights are installed at strategic locations to permit fish collection activities to continue safely during hours of darkness, if necessary.

Fig. 16 shows the Upper Baker fish collection barge just prior to its launching in the reservoir. Fig. 17 shows the barge in operation in the reservoir.
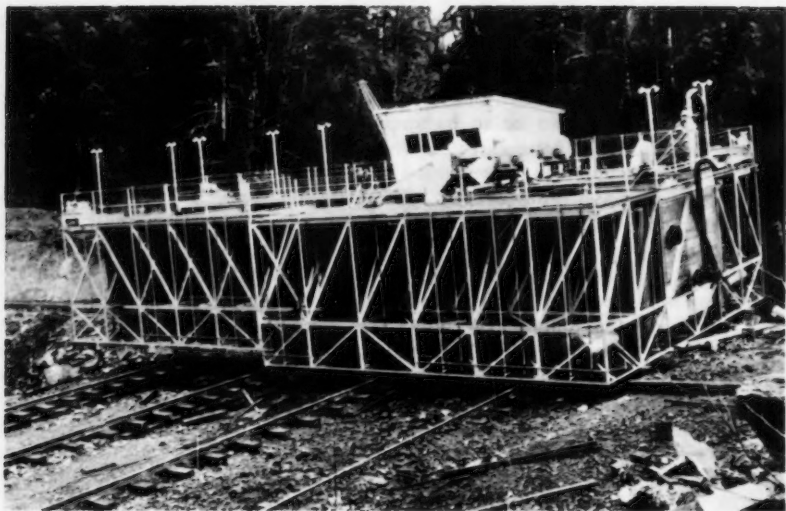
FIG. 16.—UPPER BAKER FISH COLLECTION BARGE ON LAUNCHING RAMP

FIG. 17.—UPPER BAKER FISH COLLECTION BARGE IN OPERATING POSITION
IN RESERVOIR WITH FLOAT SUPPORTS FOR SUBMERGED FISH
TRANSPORTATION PIPE AT LEFT

A pipeline of varying size is used to transport the fingerlings between the steel hopper at the fish barge stern and the tailwater pool below the dam. This pipeline has a total length of about 1,000 ft and covers a vertical fall of about 290 ft between normal headwater and tailwater levels. A general plan of the fish transportation pipe at the Upper Baker dam is shown in Fig. 18 and an elevation of this pipe is shown in Fig. 19.

The upstream section of fish transportation pipe is submerged and consists of about 300 ft of 10 in. inside diameter rubber lined flexible pipe installed on a slope of about 5/16 in. per ft, with the upper end located about 7 ft below the surface of the reservoir and the lower end about 15 ft below the surface. With a normal transportation flow of 600 gpm, this pipe is flowing only partly full, and vent pipes are installed at both ends to maintain atmospheric pressure inside the pipe and prevent surges. To offset the buoyant condition of the pipe when it is either partially filled or empty, steel counterweights are attached to the pipe at about 2 ft intervals. The counterweighted pipe is in turn supported at 8 ft intervals by 1/2 in. diameter wrought iron chains attached to timber framed Styrofoam floats.

The downstream end of the 10 in. fish pipe is reduced to 8 in. by a steel pipe reducer and connected through a 90° elbow to a vertical section of 8 in. inside diameter rubber lined flexible pipe. The upper end of the 8 in. pipe is supported by chains from a steel framed guided float which operates on guide rails attached to the upstream face of the Upper Baker dam. The lower end of the 8 in. pipe is connected through an 8 in. by 12 in. pipe expander to a 12 in. diameter steel pipe embedded in the dam. It is arranged in such a manner as to form a large loop at the bottom of the flexible pipe. The water containing the fingerlings is discharged from the lower end of the 10 in. horizontal pipe as a free falling stream and drops vertically inside the 8 in. pipe for a distance of about 60 ft before reaching the standing water surface at the bottom of the pipe, as indicated in Fig. 19.

The flow of water in the transportation pipe is controlled by a motor operated plug valve installed in the 12 in. pipe that passes through the dam. This valve is located in a valve chamber formed in the mass concrete of the dam and is operated in either a fully open or fully closed position. Water discharged from the valve enters an 18 in. diameter steel pipe which is supported on the downstream face of the dam on special pedestals and laid on a slope of 20° from the horizontal. The transportation flow containing the fingerlings is accelerated as it moves down the inclined pipe and is finally discharged into the tailrace channel below the dam at a velocity estimated to be as high as 40 fps. Fig. 20 shows the 18 in. pipe installation.

The general arrangement and type of facilities used to collect and transport downstream migrant facilities at the Lower Baker dam are very similar to those just described for the Upper Baker dam. The fingerlings are collected in a barge moored in the reservoir upstream of the dam and transported by pipe between the barge and a point below the dam where they are discharged and allowed to drop vertically about 160 ft into the tailwater pool. A schematic plan of this installation is shown in Fig. 21.

INTAKE FISH BAFFLE

Because power requirements may make it necessary to draw down New Baker Lake by as much as 50 ft, below full pool level, a means had to be pro-
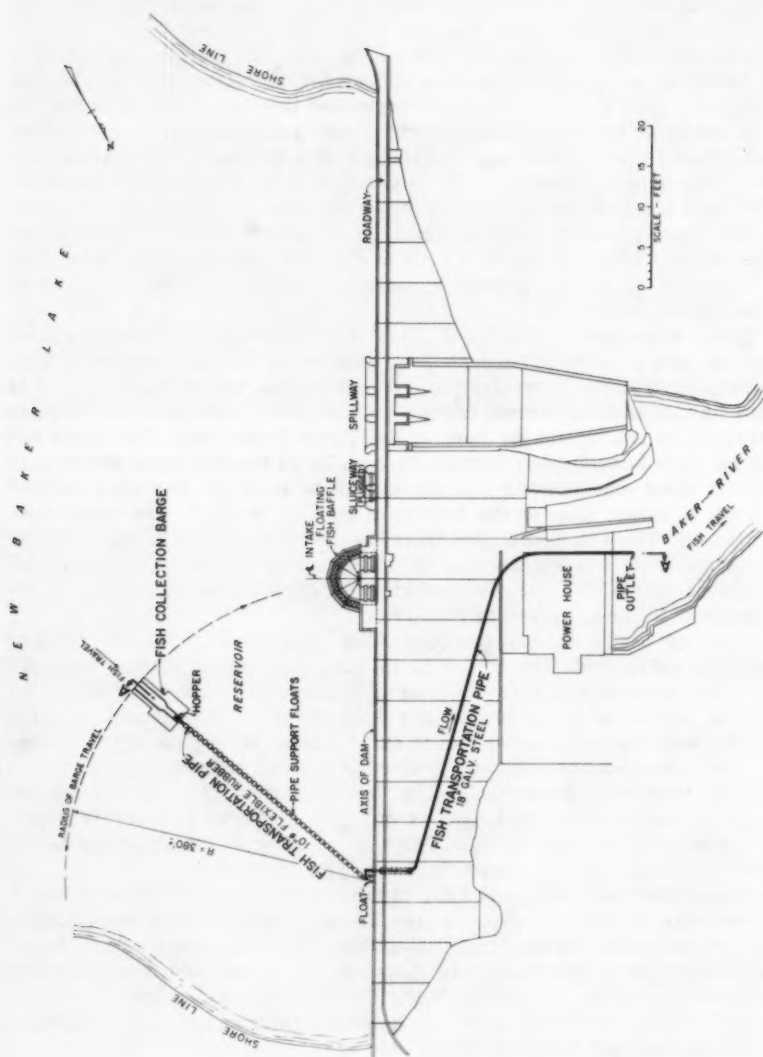
FIG. 18.—GENERAL PLAN OF FISH TRANSPORTATION - UPPER BAKER DAM
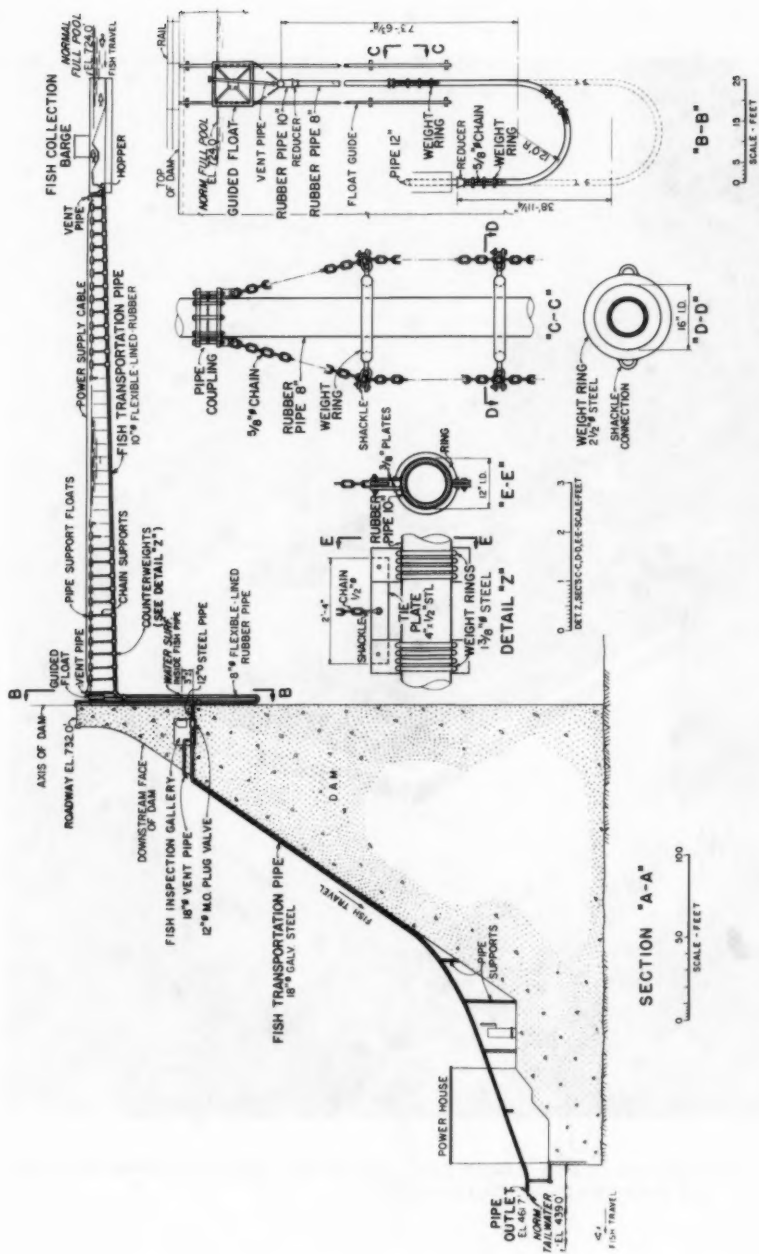
FIG. 19.—FACILITIES FOR FISH TRANSPORTATION – UPPER BAKER DAM
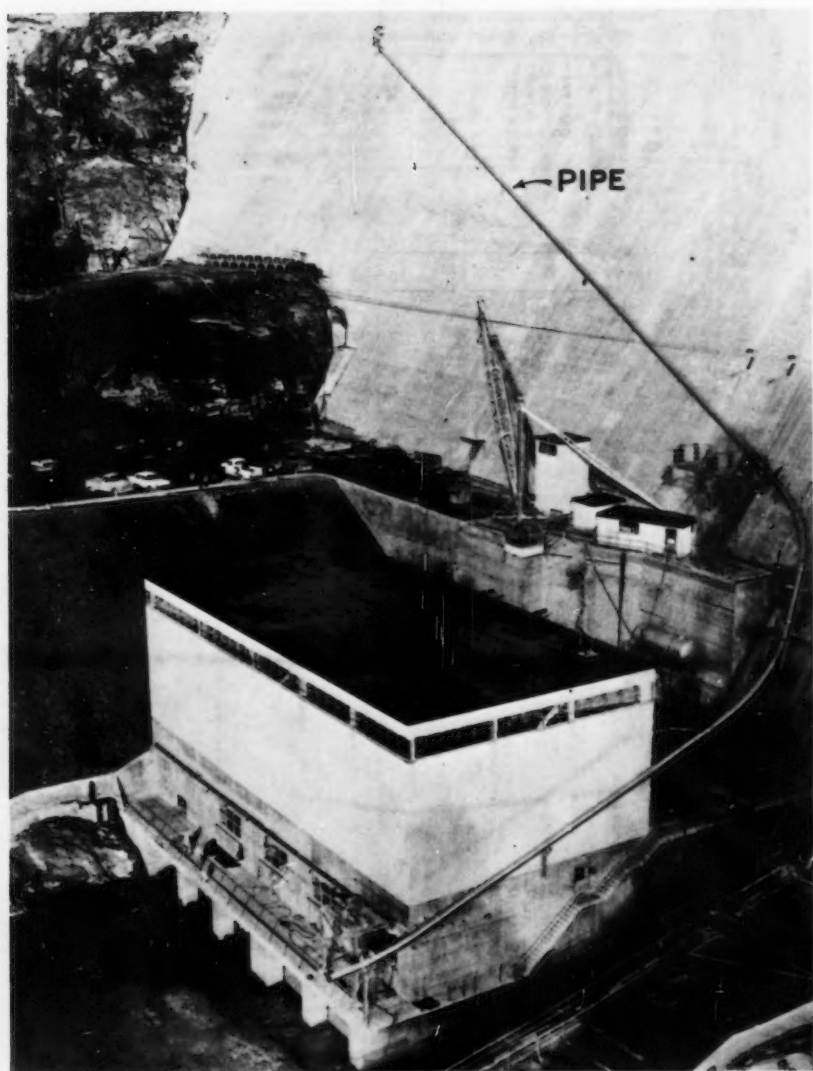
FIG. 20.—VIEW OF UPPER BAKER DAM AND POWER HOUSE SHOWING SECTION
    OF FISH TRANSPORTATION PIPE

FISH COLLECTION BARGE

Fish Travel EL.

HOPPER

VENT PIPE STEEL

PIPE SUPPORT FLOATS 33" O.D. STEEL (TYPICAL)

6" (TYP.)

1/2"ø W.I. CHAIN

6" (TYP.)

Fish Travel

FISH TRANSPORTATION PIPE 10"ø FLEXIBLE LINED RUBBER

120'-0" N.T.S.

M.O. HOIST

VENT PIPE STEEL

2"ø STEEL BAR

1/2" PLATE

1/2" CHAIN

5/16"ø BOLT

10"ø I.D.

TYPICAL WEIGHT RING DETAIL.

0   2
SCALE-FEET

NOTES:
(1) NORMAL FISH TRANSPORTATION FLOW IS 600 GPM
(2) PIPING ARRANGEMENT PERMITS OPERATION OF FACILITIES UNDER MAXIMUM RESERVOIR DRAFT OF 40 FT.

HOIST FLOAT STEEL

1/2"ø W.I. CHAIN

NORMAL RESERVOIR EL. 438.6'

1/2"ø WIRE ROPE CABLE

TRAVELING GATE HOIST

DECK EL. 446.15'

CREST GATES

FIXED CREST EL. 423.3'

Fish Travel

8"ø FLEXIBLE LINED RUBBER PIPE

WEIGHT RING (TYPICAL)

1/2" CONTINUOUS CHAIN

W.S. INSIDE FISH PIPE EL. 383.6'

8"ø STEEL PIPE

10'-0"ø

8" M.O. PLUG VALVE

VENT PIPE

VALVE HOUSE

DAM

0   10   20   30   40
SCALE- FEET

FISH TRANSPORTATION PIPE 8"ø ALUMINUM

PIPE SUPPORTS 8"ø ALUMINUM (TYPICAL)

END OF FISH PIPE

WATER and FINGERLINGS DROP VERTICALLY ABOUT 160 FT TO TAILWATER POOL

Fish Travel

GROUND SURFACE EL. 348(±)

FIG. 21.—FACILITIES FOR FISH TRANSPORTATION – LOWER BAKER DAM

vided for preventing fish, particularly migrating fingerlings, from entering the intakes for the hydraulic turbines during low water periods. After considerable study of various intake arrangements, it was decided to construct a semicircular baffle directly in front of the two intake openings. The general arrangement of the intake fish baffle is shown in Fig. 22.

The fish baffle is an aluminum structure that provides a curtain extending to a depth of 100 ft below the water surface and is supported on two lines of floating pontoons. The inner line of pontoons consists of six box shaped aluminum floats, 3 ft-4 in. wide by 2 ft-2 in. deep, and approximately 14 ft long. They are bolted together forming chords of an arc of a circle. The outer line consists also of six box shaped floats, 3 ft-5$\frac{1}{2}$ in. wide by 2 ft-2 in. deep, and approximately 20 ft long. They are similarly bolted together. The lines of pontoons are spaced about 7 ft-4 in. from each other. They are furnished with handrails along the outer edge of the outer pontoons and along the inner edge of the inner pontoons. A timber fence or fender is provided along the outer side extending about 8 ft below the water surface to protect the outer pontoon and the upper part of the baffle from possible damage by boats, logs or floating debris. Each pontoon section is divided into three watertight compartments, and each compartment is provided with a 6 in. diameter inspection hole and cover plate.

The baffle, or curtain, consists of vertical sheets of corrugated aluminum V-beam siding fastened to horizontal structural aluminum trusses. There are 21 of these horizontal trusses spaced about 5 ft apart vertically to form the 100 ft height of baffle. Each truss is a 3-hinged arch, one hinge being at the crown, or center, while the other two are at the guide bushings and roller sets. These transfer the truss reaction to the vertical guides on the dam permitting the baffle and pontoons to follow changes of reservoir water level.

Forty panels, each containing fourteen 8 in. by 36 in. wide flaps, are provided in the aluminum siding to allow water to discharge during load rejection. This prevents a build-up of unbalanced hydrostatic pressures on the thin curtain wall.

The guides on the upstream face of the dam consist of round stainless steel bars, carefully aligned to form two straight tracks and of sufficient size to resist safely and expected loads that may be applied to it by the baffle.

A nylon cord trash net is attached across the bottom of the fish baffle. Its function is to prevent water logged timbers, tree limbs or trunks, or similar trash from being drawn up through the baffle and into the intake. The net is made up of 3/8 in. diameter solid braided nylon cord with a 6 in. square mesh. The lines are looped over aluminum pulleys every 12 in. to connect the net to the baffles.

CONCLUSIONS

The total cost of the new Baker River fish handling facilities is approximately $2,000,000 or about 4% of the total project cost of $50,140,000. This appears to be a favorable cost ratio when compared to similar hydroelectric developments constructed in recent years in the Pacific Northwest.

It is also significant to note that with the automatic, or semiautomatic, type of equipment that has been provided for handling fish at this project,
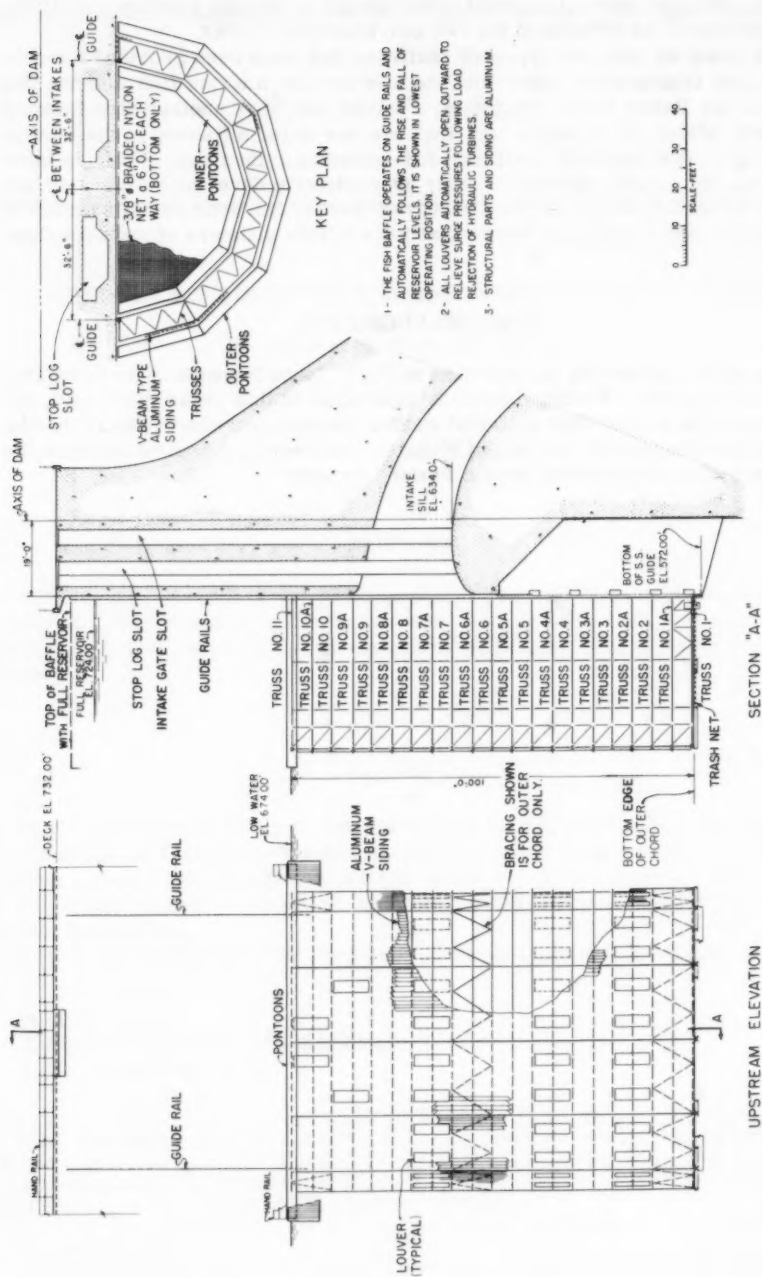
FIG. 22.—GENERAL ARRANGEMENT OF INTAKE FISH BAFFLE - UPPER BAKER DAM

only two men are normally needed to operate all of the fish handling facilities. They also serve as drivers of the two fish transport trucks.

It is believed that the types of facilities that have been provided for collecting and transporting upstr am and downstream migrant fish past the two dams of the Baker River Project are unique and mark another step forward in man's effort to conserve and improve the salmon fishery in the Pacific Northwest. The general methods and principals involved could, in most instances, be readily adapted to other hydroelectric developments of medium to high head and would afford a relatively simple and economical solution to many of the fish handling problems that now confront planners of such developments.

ACKNOWLEDGMENTS

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CURRENT DEVELOPMENTS IN HYDRO-ELECTRIC PLANT DESIGN

By William H. Wolf,[1] F. ASCE

SYNOPSIS

A resume of current (1961) developments in hydro-electric plant design is presented. The scope of design is reviewed as it affects the site studies, the plant layout, the structural problems, and the preparation of costs and estimates.

INTRODUCTION

During the past 15 yr since the publication of a paper by Palo and Marks[2] on the design of hydro-electric stations, several important developments in the civil-structural field of this work have occurred. The purpose of this paper is to review them with the intent of consolidating our gains and encouraging further discussion.

The developments will be reviewed under four general headings, namely:

1. Site studies
2. Plant layout
3. Structural design considerations
4. Costs and estimates

It should be reaffirmed that hydro-electric plant design is not the sole province of the civil or structural or mechanical or electrical engineer. The specialization inherent with the progress in each of these fields during the past 15 yr has precluded any possibility of this. No individual engineer lays out an economical plant today. Rather, it is a group whose efficiency depends on an understanding and appreciation of each other's problems. In the design activities of the United States Bureau of Reclamation (USBR), Department of the Interior, it is the province of the civil-structural engineer to coordinate these combined efforts and complete the drawings that represent a plant arrangement. It is this viewpoint and experience that are primarily discussed herein.

SITE STUDIES

With the site of the dam established, the powerplant can be arranged as a part of the dam, adjacent to the dam, or even miles downstream from the dam. In every case, the power waterway (tunnels, penstocks, tailrace) should play the most important part in the selection of the plant site. It is not that the waterway is always a more expensive or difficult feature, but for the development of the available head of water the adoption of the shortest feasible waterway normally develops into the most economical project. Although this may have been a well-established assumption in the past, it is axiomatic today because of the wide variety of economical plant layouts that have been developed. These have been made possible through the use of new excavation methods, foundation treatments, construction techniques, equipment arrangement, and handling facilities. As a result, the planning and site selection can well be done with the thought that the powerplant can be economically adopted to almost any feasible waterway.

In selecting the power waterways that are to be considered for the site studies, current practice has established several criteria: First, the waterway should be as short as possible for the development of the available head of water. Second, separate waterways for each unit are often most economical. Third, when separate waterways are not feasible, the manifold and turbine guard valves should be located as high along the waterway profile as possible. Fourth, open-cut tailrace excavation today is often less than the penstock cost.

The shortest, most direct waterway can usually be developed into the most economical arrangement, regardless of whether the plant is on the surface or underground. It often saves construction time and material, eliminates the need for surge tanks, and minimizes frictional head losses. An early determination and common knowledge of the value of 1 ft of head on the project will do much toward retaining the proper perspective throughout these studies. The location of the switchyard should never influence the attempt to obtain the shortest feasible waterway, as switchyard costs per kilowatt are relatively small.

The advantages of separate waterways for each unit are best illustrated when the powerplant is planned to be at the dam-site. With the low-head developments and gravity dams, the location of the plant at the toe of the dam is a routine matter. The sites for high concrete arch dams usually leave a location at the toe of the dam that is restricted by the canyon walls. However, alternate studies will invariably show that separate waterways through the dam to a plant at the toe are more economical than a plant location on the abutments. Although this arrangement across the canyon may incur placement of

large quantities of backfill concrete in the river channel, the shorter separate waterways eliminate the penstock manifolds and turbine guard valves normally required to serve an abutment location. If the canyon walls prohibit a conventional straight-line layout of the powerplant at the toe of the dam, several alternate plant arrangements are possible that may permit utilizing this site. For example, the service bay and control bay or both can be moved downstream to give an L- or U-shaped layout. The control bay may also be located on the roof of the unit bays or service bay to permit moving the plant closer to the toe of the dam, thereby shortening the penstocks.

Another solution may be to curve the plant to match the arch of the dam. A curved layout is perhaps a misnomer as the unit bay construction would be wedge-shaped, similar to arch dam blocks, resulting in segments that approximate a circular curve. The only curved line required would be the crane rails. This would entail a more costly overhead crane or gantry, but may well be offset by reduced costs from shorter penstocks. However, the several studies of curved layouts made by the Bureau of Reclamation have not shown savings significant enough to warrant their adoption.

If the plant must be located on an abutment, a careful study will often show that separate waterways for the units are the preferable solution when manifolds and guard valves can be avoided. A particular aid in this type of site study is the fact that contemporary practice indicates that fewer and larger units will prove more economical than a larger number of smaller units for a given plant capacity. There are two basic reasons for this conclusion. First, with the integrated power system network that currently covers our country, the need for additional flexibility (more units) to meet the local demand load has been minimized. Second, the physical size of the unit and the hydraulic features do not vary linearly with unit capacity. The size of the unit varies with the area of the required waterway because, for a given plant capacity, the head and total discharge remain the same.

For example, a 300,000-kw plant with four 75,000-kw units will require less overall length than one with six 50,000-kw units. Although width and height will be slightly larger for the four unit 75,000-kw plant, the plant cost will normally be less. Unit size, therefore, should be the largest feasible to meet equipment or shipping and handling limitations, provided no difficulties are anticipated with the larger penstock sizes.

If one or two power tunnels with their attendant manifolds and guard valves prove more economical than separate unit waterways for a multiunit plant located on the abutment, consideration must be given to locating the manifold and guard valves as high as possible on the waterway profile. There is no measurable operating advantage to having the valves located within or adjacent to the powerplant, and a considerable reduction in their cost follows from the smaller operating head for which the valves and manifold must be designed.

Open-cut excavation has not increased in cost in the same proportion as the penstock costs during the past 10 yr or 15 yr because of the developments in earth and rock moving equipment. As a result every plant site on an abutment, or at the end of a penstock located on a hillside, warrants an early check before the foundation investigations are well under way. The simplest expedient of balancing penstock costs per foot against tailrace excavation per foot will often move the plant upstream a surprising amount. Although the deeper cuts may seem formidable, the access problems are usually easy to reconcile so that the estimated savings from this waterway arrangement may be retained.

Underground schemes are usually investigated at each site where the geology warrants their application. Here again, the feasible arrangements of the power waterway are the basis for the studies. The four basic waterway arrangements for the development of underground hydro-electric schemes are clearly discussed in a paper by F. L. Lawton.[3] The essential elements that comprise "head" and "tail" developments as well as his considerations of the relative economics of surface and underground plants, their maintenance costs, their operating experience, and the comparative data assembled, are the most comprehensive current review known to the writer of this paper.

The greatest measure of economy with the underground studies is achieved by the use of one of these four waterway arrangements that are inherent to all underground schemes, not simply the use of an underground powerplant. With sound rock, an economical layout for the powerplant is possible. However, the only structural resemblance between an underground and surface station should be in the common use of concrete and reinforcement. The differences will be discussed in detail under the heading "Layout."

In the last 10 yr to 20 yr, there has been a definite trend toward specialization in the different engineering fields. This has been prompted by the rapid advances in our technology. However, although the specialization has resolved many of the technical problems, it has made the coordination of the design effort more complex. The coordination is particularly important during the site studies because the major decisions committing the expenditure of the project money is made at this time. These decisions are necessarily made at the upper administrative level and ultimately reflect the stature of the organization. Currently, the coordination required after the site studies are completed is usually assigned to a "project manager" or the equivalent. The need for this is almost universally recognized, but the extent of his responsibilities and title vary with the job and organization.

The comparative estimates made during the site studies are a necessary aid in evaluating the alternate schemes. To utilize them intelligently and to minimize the time and money needed for their preparation, a dollar increment should be set to delineate the degree of refinement desired. This may be arbitrarily established at 2% to 5% of the total scheme cost or a fixed value such as $200,000 may be used. Then, any variable which affects the final scheme less than $200,000 should not be considered during the site studies.

This simple expedient will concentrate both time and effort on the matter at hand, and it will avoid needless discussion of such variables as indoor or outdoor plants until the layout studies of the plant are begun. The experience and judgment of the administrator will be used, and, of course, the estimates may be made more quickly and economically.

PLANT LAYOUT

Palo and Marks established the principles for an intelligible approach to plant layout. Although their paper[2] was keyed to the low-head TVA powerplants, the same basic approach may be generally used and applied to medium- and high-head developments. However, in contrast to the postulates of Palo and Marks, which might be construed as emphasizing the horizontal flow of

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3 "Underground Hydro-Electric Power Stations," by F. L. Lawton, The Engineering Journal, Vol. 42, No. 1, January, 1959, p. 3, p. 67.

power, controls, and auxiliaries, the writer prefers a "vertical" conception. That is, each unit bay is an entity, as are the service and control bays, and each is laid out primarily to perform a specific purpose. Modifications to the overall size of a bay should never be dictated by anything other than the primary items which they serve.

Layout of Multiunit Surface Plants.—The unit bay layout, the most expensive feature and therefore the most important, is predicated on five primary items to be considered in the following order.

1. The plant waterway (turbine, draft tube, and penstock connection)
2. The generator
3. The main power transformers
4. The overhead cranes or equipment needed for installation and maintenance.
5. The maximum tailwater.

Considered in this order these items offer a sound basis for arranging a unit bay, inasmuch as one proceeds from the foundation upwards—a natural precept of a structural engineer. The first four items only should be considered in alloting space within a unit bay for maximum economy. Once these space requirements are established and the plans and sections drawn around them, the mechanical control, electrical control, and the auxiliary equipment can be located in the space inherent in the required framing. If not, they are more properly located in a structure designed for the specific purpose, namely, the service bay or control bay. The only economical departure from this arises in plants of one or two units in which widening the machine hall may permit elimination of the service bay or control bay and result in overall economy. With three or more units, true economy will accompany a minimum unit bay arrangement.

As an illustration of the foregoing, take the following example of a multiunit plant having vertical-shafted reaction turbines. The plant waterway outline, generators, transformers, and crane are determined by the respective specialists or with the aid of the respective equipment manufacturers, and an assembled layout is required for estimating or further refinement into a specification drawing. Beginning with the plant waterway, the limits of the substructure can be quickly sketched as shown in Fig. 1. The desired access to the turbine pit and support for the generator establish the generator elevation as shown in Fig. 2. If space is available, either upstream or downstream, the transformer bank should be located here to minimize the high cost of the low voltage leads, Fig. 3. After the generator rotor and shaft are located in a position that would permit moving it to and from the service area for installation and maintenance, the overhead crane is located and the superstructure drawn around it, Fig. 4.

Variations in the generator (umbrella or suspended), in the transformers (a single-phase bank or three phase), or in the plant waterway (a guard valve may be required, or if a low head plant, an intake and emergency gates) should be made the basis for another sketch if they are not easily incorporated. In short, the layout should not be forced if one of these four controlling features are changed or alternatives proposed. In no case has the writer found that a unit bay should or need be increased to accommodate other equipment or auxiliaries when the space requirements of these first four items are met.
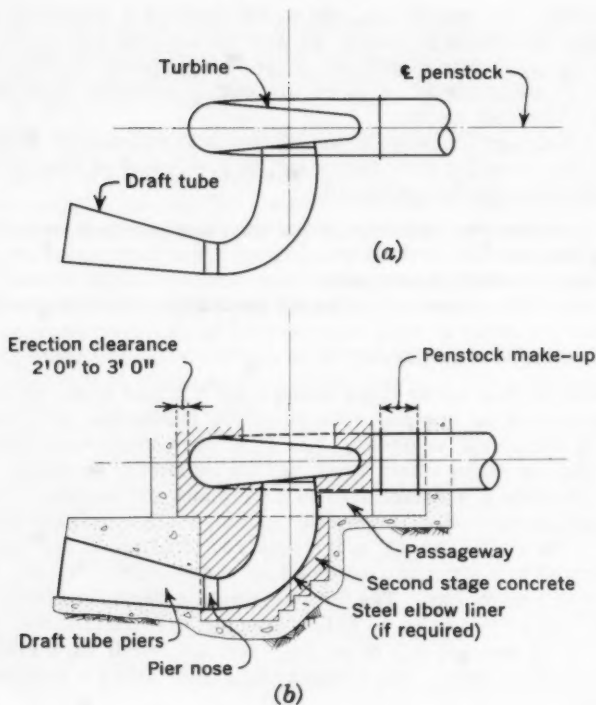
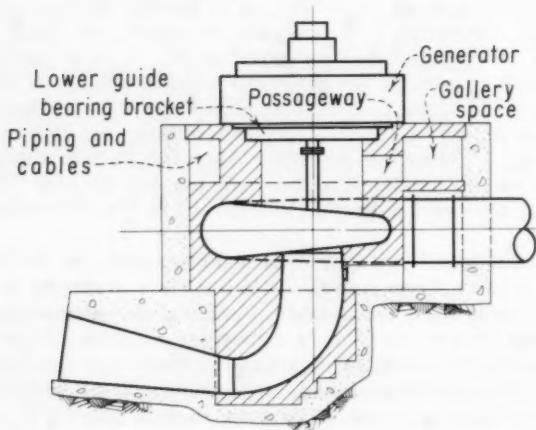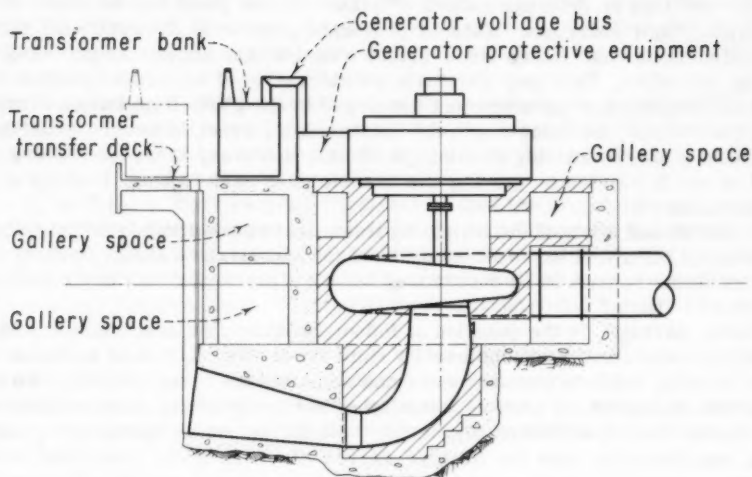FIG. 1.—LIMITS OF THE SUBSTRUCTURE

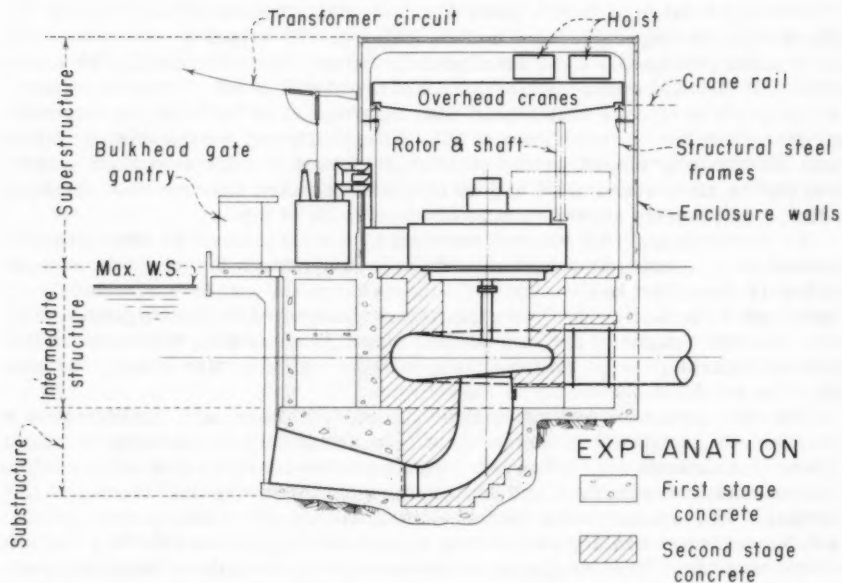FIG. 2.—GENERATOR ELEVATION

FIG. 3.—TRANSFORMER BANK

FIG. 4.—OVERHEAD CRANE AND SUPERSTRUCTURE

After this section is drawn, it should be checked against the maximum tail-water elevation to determine whether access to the plant can be made at the generator floor elevation. This is preferable, but with the multiunit plants, one often finds that future flood stages require that access be provided at a higher elevation. This may eliminate consideration of an outdoor plant or suggest the semioutdoor arrangement developed by the TVA. If an indoor arrangement is desired, entrance above the maximum tailwater is usually provided at the service bay by raising or adding a floor. However, these provisions may well result in raising or moving the transformer bank to keep it above maximum tailwater.

A sketch and study of the foregoing items leaves the cross-section and plan of a layout for a unit bay well established for comparative study. Several secondary items remain to be determined before a particular unit bay layout may be termed "firm."

First, perhaps, is the question of indoor, outdoor, or semioutdoor. Maximum tailwater considerations usually limit these alternatives to no more than two, namely, outdoor versus indoor and semioutdoor versus indoor. No general rule should be, or need be, established for the selection of an outdoor versus indoor station. Climate, protection from falling rocks, appearance, operation, maintenance, and the desired degree of economy in first cost versus higher operating cost all must be considered. Inasmuch as the difference between indoor and outdoor may favor either one by about 1% or 3% of the plant cost, the final selection is best made with due regard to some of the intangibles mentioned above.

Although a large multiunit plant offers the best opportunity for a saving by eliminating the superstructure and replacing it with a gantry, the contemporary metal panel walls have done much to reduce this differential. Masonry block will usually be less than the insulated metal wall panel. The non-insulated metal panel enclosure wall should also be considered because the alternate outdoor plant has no enclosure at all. These different wall panels, together with the highly developed, welded steel rigid frames, provide an enclosure system that is more economical and more quickly erected than the cast-in-place reinforced concrete superstructure of 10 yr or 15 yr ago.

An unusually high tailwater or deep cut at the plant site will favor consideration of a semioutdoor arrangement. A semioutdoor layout made without either of these two basic criteria is unnecessary and can be surpassed by an indoor plant because of the less expensive prefabricated enclosure panels. The economic advantages of the semioutdoor plant, so completely discussed in the Palo and Marks paper,[2] still hold true with the high tailwater usually encountered on low-head plants.

The unit electrical and mechanical control equipment and auxiliaries are conveniently positioned in the available floor and gallery space in the unit bay. These unit controls for the large multiunit plants usually consist of the actuator cabinet, pressure tank, and servomotors for the mechanical control of the turbine. The unit excitation control, unit auxiliary AC and DC power boards, and the generator braking and jacking equipment are primarily for the control of the generator. This equipment is often located in the gallery below the generator floor on either the upstream or downstream side of the unit bay. In the past, this has entailed one operator for this control area because the machines were started, stopped, inspected, and serviced by him. Today the trend is to eliminate any operator in this gallery because starting and stopping the unit,

together with the remaining important functions, are done from the control room. The unit may, of course, be started and stopped from this gallery station in an emergency, during initial testing, or following a maintenance checkout.

The remaining gallery space below or at this level can be conveniently used for piping runs, cable trays, cooling water auxiliaries, unwatering provisions, access to the spiral case, and access to the pit liner for maintenance work on the turbine runner.

It will be noted in the foregoing illustrations that the layout is made to permit installation of the equipment as a second stage in the powerplant construction. That is, a skeleton unit bay is built in which the turbine spiral case is embedded in second-stage concrete in lieu of installing it concurrently with the first-stage construction. Which alternative construction to use is a matter of economics and programming.

If the plant is to be built concurrently with a high dam, the skeleton bay and second stage installation will usually be more economical. If the plant serves as part of the dam, as on many low-head installations, the second-stage construction may prove more costly. The answer lies in reconciling several variables, namely:

1. Availability of water for power
2. Interest rates
3. Provision for future units

With a high dam, for example, the water will usually not be available for power for some extended interval, perhaps 3 yr to 4 yr after the beginning of construction. As the contractor for the dam normally constructs the powerplant concurrently with the dam, second stage construction often permits deferring purchase of the turbines for many months so that the installation of the equipment need by programmed only to meet the time when water will be available. The interest on the deferred equipment contracts will normally exceed the additional plant cost entailed by the skeleton construction. Further, in a multiunit plant, the skeleton construction will obviate the need for subsequent cofferdams if some of the unit installations are planned for a later date, as at Grand Coulee Powerplant where the last units were installed 10 yr after the first unit was on-the-line.

One decision that must be made by the civil-structural engineer in the early stages of the layout is the expansion-contraction joint system deemed necessary for the unit bay monoliths. For the smaller plants with 1 or 2 units that usually total less than 150 ft to 200 ft in length, no expansion-contraction joints are necessary. The multiunit layouts, however, pose an interesting problem in that two unit bays may often be combined into one monolith. A brief look at a typical plan at the turbine elevation shows why this may be desirable; namely, a saving in building length of 3 ft to 5 ft for every pair of units is often possible through elimination of one of the cross-walls, Fig. 5. This may be a highly desirable advantage in a narrow canyon or a site where good foundation rock dips steeply away. However pleasing this saving may be at first glance, another factor must be considered before the decision is made. Combining the unit bays into one monolith entails constructing them together. As the two bays will be designed as one structure, the horizontal and vertical construction joints must be planned and the individual placements made to insure a monolithic structure. This entails overlapping placements that force the simultaneous

construction of the two unit bays. Although some degree of stepped construction is possible so that formwork may be reused, complete duplication of the construction of each bay is not possible. A tight schedule where the construction of the plant is the controlling feature may well define whether the overall economy lies with the shorter building or shorter construction period.

The service bay and erection area are located at one end of the unit bay arrangement in a multiunit plant. Access to the plant is usually provided at this point as well as space for station auxiliaries. The powerplant sump establishes the lowest elevation—usually a few feet below the unit bay draft tube floors so that they may be drained to the sump. The access road elevation, located above maximum water surface, sets the height of the uppermost floor. The width of the service bay is necessarily identical with the unit bays to permit movement of the powerhouse cranes over the erection area. The length of the service bay is perhaps its one variable that has seen the most change. An old rule-of-thumb established it as twice the length of the unit bay. A sterner look at the fact that this area does not contribute directly to the production of kilowatts or kilowatt-hours has resulted in gradually reducing its length. As a result, our current rule-of-thumb for feasibility estimates establishes the length at 1 to 1 and 1/4 times the unit bay length. In the final design, this is easily checked against the space required for concurrently erecting a rotor, assembling or storing the upper guide bearing bracket, and access for the truck-trailer unit needed for delivering the component parts of the generating machines.

If a fully complemented machine shop is deemed necessary because of the remote location of the plant or lack of similar facilities in neighboring towns, the required space must be provided. However, the shop space should not be provided by extending the service bay. There are exceptions to every rule, but it is normally found that the shop area is more economically provided adjacent to the service bay. Then it can be arranged and designed as a light industrial building with its own crane rather than made a part of the heavy framing inherent in the unit bays or service bay.

The service bay might be more aptly termed the "station" service bay because station auxiliaries such as compressors, sump pumps and motors, oil purifying equipment, ventilation equipment, water treatment equipment, and transformer oil storage should be grouped in this area between the sump roof and the entrance floor. However, the controlling length is established by space required at the entrance floor for the erection and maintenance of the units, not their auxiliaries. Auxiliaries do not determine a powerplant size or shape.

The control bay or control area is usually arranged as a separate feature of a multiunit hydro-electric plant. It may be located at one end of the unit bays, adjacent to the service bay, near the center of the unit bays, or on the roof over the unit bays. The size of the control bay or area is dictated by the space required for the main control boards and access to them, either an elevator and a stairway or both. Many preferences are cited for the location of this area, such as a view of the tailrace, a view of the machine hall, on the generator floor, one story above the generator floor, at the end of the plant adjacent to the switchyard, or a central location with respect to the units. The trend is to satisfy the last of these requirements as completely as possible because it is found that many man-hours can be wasted traveling between the operating areas and the control room. A careful study is in order here because a location that will permit eliminating an operator means a saving of approximately $400,000 over a 50-yr repayment period excluding interest considerations.

This location does pose a problem of noise from the generating units that must be reconciled by controlling the noise at the source, or in the control room, or in the transmission paths between the source and the control room. The use of enclosures, barriers, and sound absorption devices in the open areas constitute the normal method of control

Recently the question of fallout protection has been added to the complexities of the plant layout. As the control room is the nerve center of a power-house, adequate protection would imply that it be located below ground or enclosed by heavy construction. However, the degree of protection for the plant will vary with the size and location of the powerhouse. This requirement should be defined before the plant layout is undertaken as provision for Category A protection[4] can seriously affect a proposed control room location and the structural framing.

A combined passenger and freight elevator is usually provided in multiunit plants to save time for operating and maintenance personnel. In addition, it is used to carry supplies, materials, and many of the small pieces of equipment necessary for maintenance work in the plant. This will often avoid removing service hatches and using an overhead crane. To utilize the investment in the elevator, provision should be made to service all floor levels. This may be difficult unless planned from the beginning of the layout. In addition, it should be centrally located for common use of operators from the control area and maintenance personnel from the service area. If the control room is located at generator floor level, the elevator can appropriately be located primarily for convenience of the maintenance personnel near the service bay and machine shop area, because their duties take them to all floor levels.

Layout of One- and Two-Unit Plants.—The one- or two-unit plants offer more opportunities for ingenuity and economic variations. Particular attention should be given to the site selection, as a check of penstock costs per foot against tailrace channel excavation per foot in the early stages of the study can minimize the drill holes required to establish plant foundation conditions, and will often locate the plant further upstream in a deeper excavation than a first estimate might indicate.

The layout should begin with the unit bay as outlined under the multiunit plants. However, after the five primary items are assembled in their respective order, the remaining layout features and their location are subject to a variety of economical arrangements. For instance, the plant may be lengthened upstream or downstream to incorporate normal service bay facilities and the control room area or both. The units are usually spaced together, but they may be separated by the erection area to facilitate excavation for the penstock tunnels or eliminate the transfer deck for the transformers. Handling facilities may be supplied by a derrick if an outdoor installation is feasible. A structural steel runway with an overhead bridge crane is often a more economical solution for one-, two-, and even three-unit plants than the conventional gantry.

One other noticeable difference between one- and two-unit plants and the multiunit plants is the location of the unit controls and the control room. One operator per shift or one "on-call" is the maximum provided at the one- and two-unit plants. To permit this, it is desirable and important to arrange the control room and operating equipment on the same floor level, preferably the

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4 "Fallout Shelter Surveys: Guide for Architects and Engineers," NP-10-2, Natl. Plan Appendix Series, Office of Civ. and Defense Mobilization.

generator floor, so that one operator can cover all points requiring frequent inspection with a minimum of movement from the control room.

No machine shop is provided, although a shop area for minor maintenance can usually be set aside near the service area. Service and erection areas in these plants should be an absolute minimum because their use is appreciably less than encountered in the multiunit plants. Auxiliary setting down space can be provided outside the plant, if necessary.

Underground Layout.—As mentioned previously, when the rock mass in which a station is to be located is so poor that it must be considered as a load rather than a building material, no economy is possible by the selection of an underground plant per se. The layout must be approached with the full assurance that the rock is available as a building material for the finished station. Of course, there are external forces imposed on this material through the formation of the opening within the rock mass just as there are external forces (wind, earthquake, and so forth) on a surface plant. The opening in the rock should be circular or eliptical in contrast to the rectangular configuration of a surface plant.

The five principal items mentioned for the unit bay of the multiunit plant must still be considered in their respective order, but the civil-structural engineer should abandon his surface plant framing concepts if he is to enclose the equipment intelligently. The plant waterway as previously done will serve as a beginning.

First, with the realization that no lateral stability problem exists, the base of the plant can be minimized as we enclose the waterway. The framing above the draft tube now has one primary purpose; namely, support of the vertical loads, whereas, the surface plant necessitated framing to resist both the horizontal water and the vertical loads. The horizontal water loads that may arise from seepage through the rock are provided for by a gutter system and, if unsightly, a block partition wall can conceal them. Where necessary, rock bolting should be used to stabilize the rock walls, not reinforced concrete. Gallery space may be reduced from that inherent in a surface plant because it is not dictated by the vertical framing surfaces required for an economical surface plant.

The power transformers can be located in a separate vault, or on the ground surface, or in the machine hall opposite the units. This last location is seldom preferable, as it usually increases the width of the hall excavation beyond that required for the generating units and necessitates a divider wall to minimize the fire hazard. Comparative estimates of these alternatives should be made to help establish the preferred location.

The machine hall volume is perhaps the most costly part of the layout as contrasted to the opposite relationship in a surface plant. As the controlling space requirement is installation or removal of the rotor, this should receive more serious consideration than customary. The lesser space requirements for handling the umbrella generator in lieu of a suspended type are particularly important here. However, if the suspended type is necessary because of speed, capacity, or other requirements, steps can be taken to minimize the height of crane rail. The most economical solution the writer has seen to this problem was undertaken at the Bersemis Plant in Canada. Here the distance from the top of generator housing to the top of crane rail was held to about 16 ft as contrasted to surface plant installations where it varies between 30 ft and 40 ft for units of commensurate size (150,000 hp). This was done at Ber-

semis by lifting the rotor from its spider frame rather than by the shaft. This method is also used at Stornorrfors and Harspranget and is often used by European generator manufacturers. The resulting economy is enlightening as one reduces volume of rock excavation instead of volume of wall concrete as in a surface plant. For instance, a 15-ft reduction in height of a 60-ft-wide by 500-ft-long machine hall for an enclosed surface plant would be approximately:

15 ft (2 ft x 500 ft + 2 ft x 60 ft) = 16,800 cu ft

@ $4.50 = $75,000 (12-in. cast-in-place reinforced concrete wall)

Whereas, the underground station would show:

15 ft x 60 ft x 500 ft = 450,000 cu ft

@ $1.00 = $450,000

The columns supporting the crane girder for the overhead bridge crane need only support the crane, and their lateral stability is usually assured by doweling to the rock walls. The flexibility offered by the fact that they need not be located directly opposite one another in the machine hall affords a better utilization of floor space.

With the width and height of the hall established by the foregoing requirements for handling and the bridge crane, the remaining feature to be resolved is the roof. An arch is the natural form for the roof to take from a stress standpoint and the practical one of excavation.

The arch roof is lined with concrete for the length of the machine hall in the majority of the 300 underground stations in the world today. The loads for which the concrete arch may be designed offer a problem best solved by rationalization and experience with the rock in question. Mining excavations in the proximity, photoelastic tests, an exploratory adit, and rock behavior theories can all contribute to a solution. For estimating purposes, the writer prefers the following approach.

With the machine hall roughed out as discussed previously, an ellipse is drawn, outlining the proposed excavation, Fig. 6. The ratio of the major to minor axis will usually be about 3:2. The body forces that tend to fill this space in the rock mass arise from the weight of rock above the opening. Assuming a Poisson's ratio of 0.25 for the rock, the stress distribution at the crown of this ellipse is $\pm$ zero.[5] Assuming the rock has no appreciable tensile strength, this curve would represent the upper limit of the potential rock fall into the opening.

The concrete arch can then be drawn to permit operation of the overhead crane and designed to carry the weight of rock between the arch and the elliptical opening.

Of course, this stress condition at the crown of the ellipse will alter if inherent stresses exist or the rock behaves plastically rather than elastically. It also assumes that the maximum shear stress never exceeds the shear strength of the rock. This will normally be true at the depths most machine halls are located even though the strength of the rock is usually weakened by cleavage and cracks.

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5 "A Review of Rock Pressure Problems," by Richard P. Shoemaker, An Introduction to the Design of Underground Openings for Defense, The Colorado School of Mines Quarterly, Vol. 46, No. 1, January, 1951, p. 127.

After the concrete arch is designed for the rock load, it should be checked for the pressure induced by low pressure grouting—probably 20 to 25 psi. To avoid external water loads in excess of these pressures, weep holes may then be drilled through the lining in the manner commonly used for tunnels and a drainage system hung from the soffit of the arch.

So far, structural and layout concepts from the draft tube through the roof of the underground plant have been discussed. In plan, or along the longitudinal length of the station, the civil-structural engineer also has a new basis for his part in the layout. In a conventional surface plant unit bay, the expansion-contraction joints are provided at each side of the unit framing system as shown in Fig. 5. In the underground unit bay, the crosswalls will not be needed to resist tailrace water loads and so they can be omitted. The expansion-contraction joint between the unit bays need not be kept a straight line, but may be offset if desired. Both of these provisions will permit a closer unit spacing and shorten the length of the station as compared with its surface counterpart.

Furthermore, if it seems desirable to supplement the rock-bolting and provide support for the sidewalls of the excavation with first-stage concrete, a
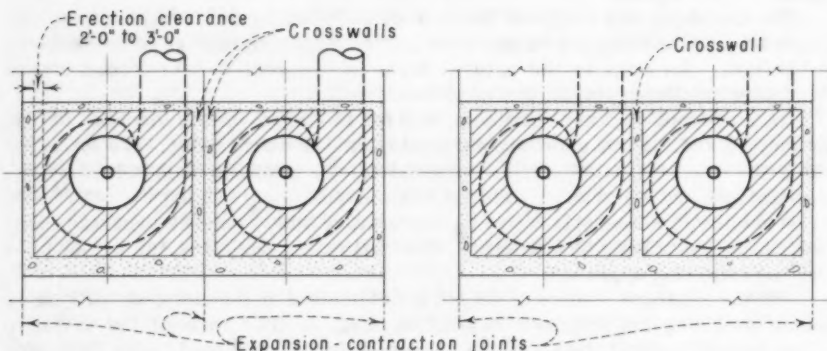

FIG. 5.—TURBINE ELEVATION

relocation of the contraction joint between each unit bay may be considered. As shown in Fig. 7(a), the usual framing can be arranged to provide a horizontal plate with an opening in the center for subsequent installation of the equipment. However, the contraction joint in the first-stage construction might be located on the centerline of the units thereby affording an I-beam shape in place of the rectangle with an opening in the center [Fig. 7(b)]. This is possible because the first-stage floors in the underground layout are primarily horizontal slabs supported by a system of columns instead of the wall and slab construction used in surface plats to withstand the external loads.

In summary, because of the absence of the lateral stability problems, the structural engineer should provide space for the equipment in a vertical rather than horizontal plane. Tangential tensile stresses around the opening in the rock mass are theoretically minimized or eliminated with an eliptical configuration. For example, if additional floor space is required, it is better to add

a floor at the generator housing level, as shown in Fig. 6, than widen the machine hall excavation.

STRUCTURAL DESIGN CONSIDERATIONS

General acceptance of the shear-friction value as indicative of the safety factor against sliding has replaced the old concept based on the direct ratio of the vertical to horizontal forces. This value is one that has been used in the
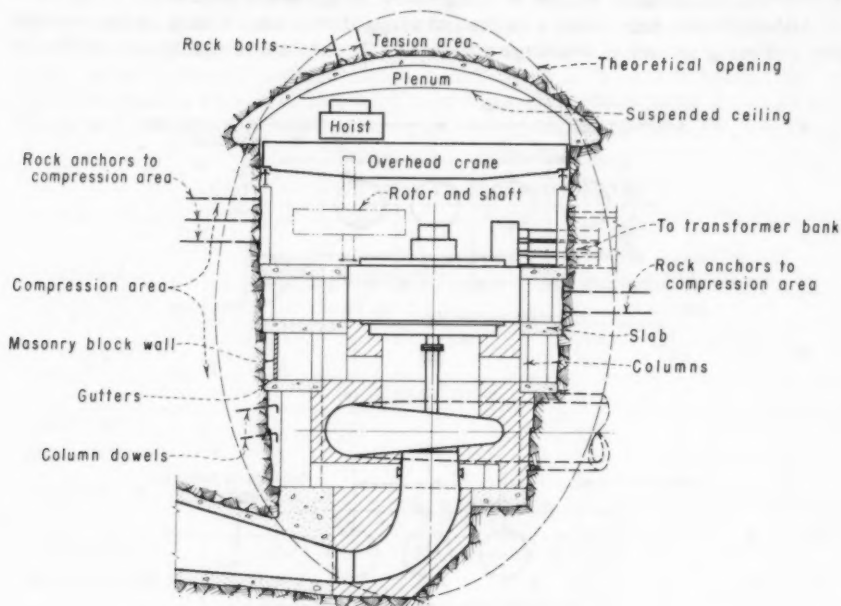
FIG. 6.—PROPOSED EXCAVATION

stability analysis of masonry dams for many years, but its utilization for structures has been somewhat slower. It is briefly represented as follows:

$$Q = \text{shear-friction factor of safety} = \frac{(\Sigma W + U)\, f + c\, A}{\Sigma H};$$

ΣW = summation of vertical forces, except uplift, Plus (+) is down;

U = uplift forces which are assumed acting over 100% of the base;

c = cohesion or unit shearing resistance applied only to area in compression;

A = area of the base in compression;

f   = coefficient of friction of the material in which the sliding may take place; and

ΣH = summation of horizontal forces.

The properties of the materials (rock, earth, concrete) are determined from laboratory tests. On the smaller jobs they are estimated from previous experience or data.

In the stability analysis the plant is checked for overturning and flotation in the same manner as in the past except that effectiveness of the uplift forces over 100% of the base, in lieu of 2/3 or 50%, is accepted practice.

Although this may seem a backward step, it has been offset by the careful use and maintenance of drainage galleries. These can be easily located in the

(a)

(b)
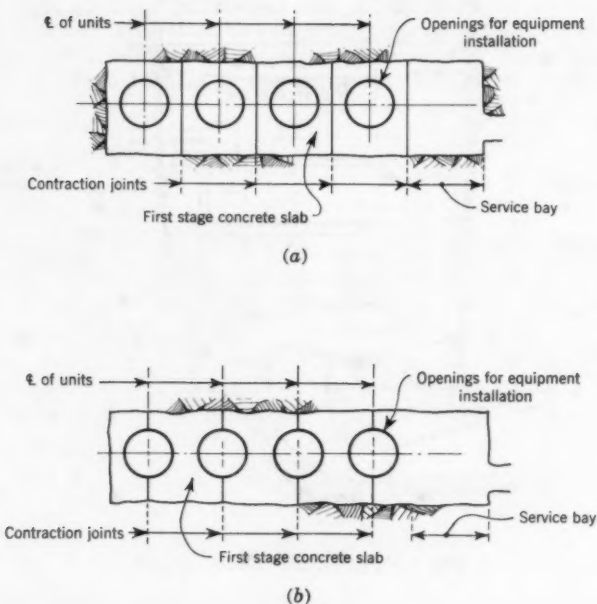
FIG. 7.—GENERATOR FLOOR PLAN

substructure of the powerplant. They must be readily accessible, lighted and inspected in the same manner as those commonly provided in the lowermost galleries of gravity masonry dams. Their intelligent use can preclude the higher cost of additional mass concrete or dowels where it is deemed desirable to improve stability conditions.

The minimum allowable factors of safety currently used by the USBR for these stability conditions are given in Table 1.

As will be noted, these values are lower than those used for masonry dams. This has seemed reasonable because of the relative damage and loss of life

entailed. However, if the powerplant is part of the dam or an extension of the dam, as found in many low-head developments, the safety factors used for the dam must govern.

The mass of concrete inherent in the substructure, which encloses the elbow-type draft tube commonly used with the reaction or propeller runner, does not lend itself to any rigorous analysis. This was carefully pointed out and discussed by Palo and Marks. No noteworthy advances in the structural analysis have been reported here. More attention has been given to the location of construction joints, the sequence of placement, the simplicity of details for the nominal reinforcement used, and the height and length of placements. For instance, when first-stage skeleton construction is used, and the draft tube elbow and spiral cases are embedded in second-stage concrete, the draft tube piers have been successfully placed in one lift (8 ft to 15 ft in height) instead of a succession of 3-ft to 5-ft lifts, and base slab placements up to 65 ft in length are customary. Substructure placements for large units (75,000 kw and

TABLE 1.—ALLOWABLE FACTORS OF SAFETY

Minimum factor of safety (1)	During construction		Structure completed and equipment operating	
	Loading		Loading	
	Normal (2)	Extreme (3)	Normal (4)	Extreme (5)
(a) Major Plants				
Overturning	1.1	1.1	1.2	1.1
Shear-friction	2.5	1.1	3.5	2.0
Flotation	1.1	1.1	1.2	1.1
(b) Minor Plants				
Overturning	1.1	1.1	1.2	1.1
Shear-friction	1.5	1.1	2.0	1.5
Flotation	1.1	1.1	1.2	1.1

up) have thereby been reduced to 8 placements instead of about 15. This permits speedier construction and reduced costs, and no detrimental cracking has been observed.

The intermediate structure of the powerplant is described as that part of the structure extending from the top of the substructure to the generator floor. This is an area in which progress is being steadily made. The first solutions were linear as typified by moment coefficients and crude approximations to reflect the monolithic structure being built. With the common usage of "moment distribution" the analysis progressed to a "planar" one wherein the approximations were correspondingly reduced.

Today (1961), with the development and application of the electronic data processing machines, a three-dimensional analysis can be used that reflects the true action of the structure under the various combinations of loadings. These computers do not affect the analysis, but they permit a more thorough

structural analysis through the solution of the many simultaneous equations which was prohibitive with a desk computer or slide rule.

This three-dimensional analysis of the upstream and downstream intermediate structure framing system is currently made at the USBR[6] through a modified trial-load analysis. The applied loads are divided between a grid of the principal horizontal and vertical elements of the structure which maintain continuity of the elements. Major horizontal elements consist of the floors and major vertical elements consist of the piers and columns. The effects of shearing deformations and yielding at the supports as well as variable sections are included. Instead of utilizing the trial process often used in the trial-load analysis, the solution of load distribution is determined by use of a system of simultaneous equations set up for solution by an electronic computer.

The analysis confirms the importance of the downstream or upstream gallery floors as horizontal beams in the framing system and encourages the elimination of beam and slab construction for the gallery floors. This will permit more flat slab construction with simpler form work and faster construction and lower costs.

Experience has shown that the use of precast or pre-fabricated materials for the superstructure will result in a more economical, more efficient enclosure than cast-in-place concrete. Currently, and for the past 10 yr, structural steel provides this answer for the framework. The main superstructure members are welded, built-up rigid frame steel bents that support the welded steel crane runway girders. The enclosure walls may be masonry units, steel panels, or precast concrete panels, or a combination of these for architectural purposes. The roof is usually a lightweight precast roof slab, timber decking, or metal decking. The structural steel bents and the prefabricated enclosure panels permit speedy erection under all weather conditions. This will often permit an earlier on-the-line date that may even defray a large part of the superstructure costs.

The design of the second-stage concrete forming the spiral case for low-head plants is covered in the Palo and Marks paper, and no significant changes are known to the writer other than the concept advanced by Charles Jaeger[7] wherein he develops the use of the barrel-shaped generator foundation as a cylinder of finite length that assists to resist the loads from the spiral casing.

In plants of medium- and high-head turbines, the spiral cases are usually of welded plate steel or cast steel and normally completely embedded in concrete. If the spiral case is erected, tested, and then emptied before placing concrete around it, the tendency of the embedding casing to expand, under the normal operating conditions, produces stresses in the surrounding concrete with resultant local cracking. This cracking is usually eliminated or controlled by one of two different measures. The first is to cover the outside of the casing with cork or other resilient material to permit some expansion of the casing without cracking the concrete. The second, which is used by the USBR, is as follows:

After the casing has been completely assembled in place in the turbine pit, a test head is attached to the inlet end and a steel test ring placed just inside the stay vanes to close off the annular speed ring opening. Before concreting,

6 "Glen Canyon Powerplant," by Samuel Judd, U. S. Bur. of Reclamation Technical Memorandum 658, Dept. of the Interior.

7 "Design of Unlined Spiral Casings for Low-Head Stations," by Charles Jaeger, Water Power, February and March, 1952.

the casing is filled with water and tested for leaks under the normal operating pressure, and it may be tested to one and one-half times normal pressure unless previously shop-tested. Concreting operations are then begun with the casing prestressed to normal operating pressure. Care is taken to keep the surface of the placement level and the rate of placement low—about 1 ft to 1.5 ft per hr—so that the casing is not shifted. Water may be circulated through the casing to minimize the temperature rise of the mass concrete surrounding the spiral case and thereby reduce cracking and eliminate distortion of the machined surfaces of the speed ring receiving the turbine cover plate.[8] These procedures permit complete embedment in one or two lifts and avoid a succession of 3-ft lifts with their delay and expensive joint cleanup. Hoop reinforcement steel is placed around the casing to further control cracking that might arise from the expansion of the casing when it is subjected to above normal operating pressures such as water-hammer. This nominal hoop reinforcement is selected by assuming it carries the increase in pressure as a hoop in tension. Admittedly, this basis for the selection of the reinforcement is theoretically incorrect; however, it has provided a successful guide for a wide
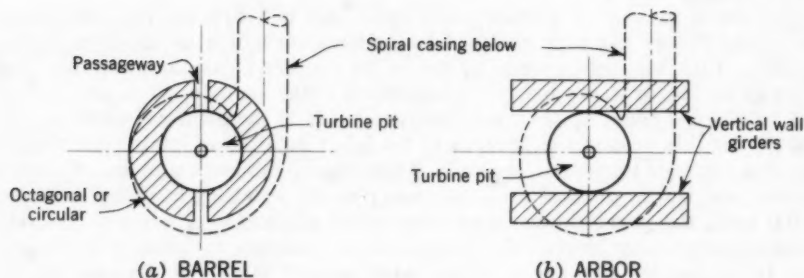

FIG. 8.—GENERATOR FOUNDATION PLAN

range of units installed over the past 20 yr. The concrete, which carries the greatest part of the load resulting from pressures in excess of the normal operating pressure, has been free from cracks on existing installations, and there has been no evidence of any shifting of the spiral casing that might result in seizure or wear of the runner seal rings.

Currently (1961), two basic types of generator foundations are used. For simplicity, they may be termed the "barrel" foundation and the "arbor" foundation. Both are located directly over the concrete embedding the spiral casing and both have advantages and disadvantages.

The "barrel" is more generally used and its basic arrangement in plan is shown in Fig. 8. It has the advantages of permitting a lower generator setting and more space outside of the turbine pit. Structurally, it is indeterminate because of the equipment openings and recesses required and the discontinuity effected by the passageways to the turbine pit. The maximum barrel thickness

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8 "Pressure Embedment of Spiral Cases," by Bradley G. Seitz, Transactions, ASME, 1958, Vol. 80, p. 115.

is usually determined by the generator housing radius less the turbine pit radius. The minimum thickness is the generator stator soleplate radius less the turbine pit radius. The height of the barrel and the lateral support afforded by intervening floors permit an intelligent estimate between these maximum and minimum thicknesses.

The "arbor" has been proposed as a more comprehensive structural element inasmuch as the two longitudinal wall girders can span across the spiral case and thereby permit a determinate analysis. The horizontal girder spanning between the vertical wall girders completes the "arbor." Because some of the generator foundation plates must be located on this horizontal girder, the generator setting is necessarily higher than that with a "barrel." However, when the plant is subjected to a high tailwater surface (above the generator floor), or if it is desired to provide for turbine runner removal in a longitudinal gallery above the spiral casing, the "arbor" affords a means of structural framing that is superior to the "barrel" with no increase in cost. The wall thicknesses for the "arbor" can be sized in a manner similar to the "barrel" examined previously. The horizontal girder must be designed to minimize any deflection at the generator soleplates. This should be checked with the generator manufacturer.

The use of precast or prefabricated materials in hydro-electric stations in the United States has been confined to the superstructure as mentioned previously. This has been primarily due to the requirements for watertightness in the galleries or areas below the maximum water surface. The use of precast reinforced concrete for the submerged parts of the plant has seemed prohibitive for this reason and because of the labor entailed in joining the myriad variable sections together. However, if the watertightness requirements were relaxed, and larger precast sections were planned, entailing handling facilities of 100 tons, the possibilities of an economical utilization of precast concrete would be measurably increased. These are new concepts for most of the American designers, but they are currently used in the USSR. For example, Soviet engineers have been steadily pursuing this idea with the result that while their V. I. Lenin Hydro-Electric Plant entailed only 2% precast reinforced concrete construction,[9] the Stalingrad Plant used 4%, and 600 thousand kilowatt Plyavin'sk Plant was increased to 35%; currently the Saratovsk Plant utilizes about 50% precast reinforced concrete construction.

Their reported savings attributed to this type of construction are due to lighter weight structures and the reduced volume of work, a reduction in construction time, and a reduction in the cost of precast construction because of continuous production of larger precast elements.

The lighter weight of the structures was attributed to reducing the factors of safety in the design coefficients of stability that "were not based on sound scientific principles."[10] The reduction in construction time was possible because precasting was carried on at the same pace throughout the year, thereby avoiding a winter slowdown. The reduction in precast construction costs by using larger elements is noteworthy because at Saratovsk it entails a skeleton struc-

9 "Means for Reducing the Cost of Precast Reinforced Concrete in Hydrotechnical Construction," by Eng. V. P. Okorochkov, Gidrotekhnicheskoe Stroitel' stvo, No. 11, November, 1959.

10 "Further Progress of Hydroelectric Construction," by Eng. V. S. Eristov, Gidrotekhnicheskoe Stroitel' stvo, No. 11, November, 1959.

ture assembly utilizing two 80-ton traveling jib cranes that installed precast sections up to 150 tons. Subsequently, precast elements weighing up to 600 tons may be assembled by the powerplant cranes. The elements are made monolithic after construction by welding the reinforcement and concreting the transverse and longitudinal joints.

These new concepts of design and construction may present an opportunity for savings that have not been rigorously pursued in this country.

Inasmuch as precasting on the scale they have found economical demands a large multiunit station and space available for the casting yard adjacent to the plant, the technique cannot be applied to all stations. The advantages may be realized if alternative designs—precast or cast-in-place—should be presented for bidding purposes. Although this would increase the engineering costs and time required for the preparation of the specifications, the net savings (as represented by USSR engineers) may be worthy of considerable engineering research and expense.

COSTS AND ESTIMATES

Costs and the measures currently taken to reduce them have been examined with the individual features throughout the paper. In summarizing, it should be emphasized again that the most useful index for comparison is the cost per kilowatt of installed or nameplate capacity (defined as the plate capacity when operating at a head at which full gate turbine capacity produces generator capacity). This is an almost universal way of describing the relative cost of a station. Moreover, this index may be used to great advantage during layout and design studies if the cost per kilowatt of the individual features are tabulated for comparison. This will constantly focus attention on the expenditures to be made for the dam, spillway, outlet works, powerplant structure, power waterway, major equipment, and so forth. The better features of comparable schemes are spotlighted, and this often encourages incorporating them and weeding out their more expensive counterpart. Refinements are more easily assessed and the administrative personnel can properly encourage or recognize new developments at their face value. An economical layout today is more apt to be the result of a succession of minor developments rather than one bold measure. Progress will consist of a forward step or new development on each job, and it is there to be made if we search relentlessly for it. Steam station designers could never have increased their efficiencies from 1 kwh per lb of coal to 1-1/2 kwh per lb of coal (12,000 BTU per lb of coal) if they had continued to use the same designs of 10 yr and 15 yr ago. Past designs must be considered as a stepping off point, not necessarily the answer.

Three types of cost estimates are currently prepared in connection with a hydro-electric development. They are called reconnaissance, preliminary or feasibility, and the engineer's estimate.

The reconnaissance estimate is defined as the one used in the broad planning stages of a project study. It should be used to help select the one or two best sites or routes from the many alternates presented. The preliminary or feasibility estimate is made to determine the best arrangement of the different features at a particular site. Although not completely detailed, it should represent an accurate capital cost figure from which the benefit-cost ratio may be established. The engineer's estimate is made prior to issuing specifica-

tions to assist in evaluating the bids for the work and to refine the particular arrangement selected.

These three estimates reflect the orderly process of site and plant selection. As mentioned before, a common error during the reconnaissance phase of the investigation is to attempt or request decisions on items that are properly left for the preliminary estimating stage. For instance, the final determination of the number of units should not be injected into reconnaissance studies, nor should the question of an indoor or outdoor plant be considered. These should be deferred to the preliminary estimate and the latter may even be handled subsequent to that.

The reconnaissance estimate for the hydroplant (FPC Accounts No. 331, 333, 334, and 335) should be taken from estimating curves if available. Fig. 9
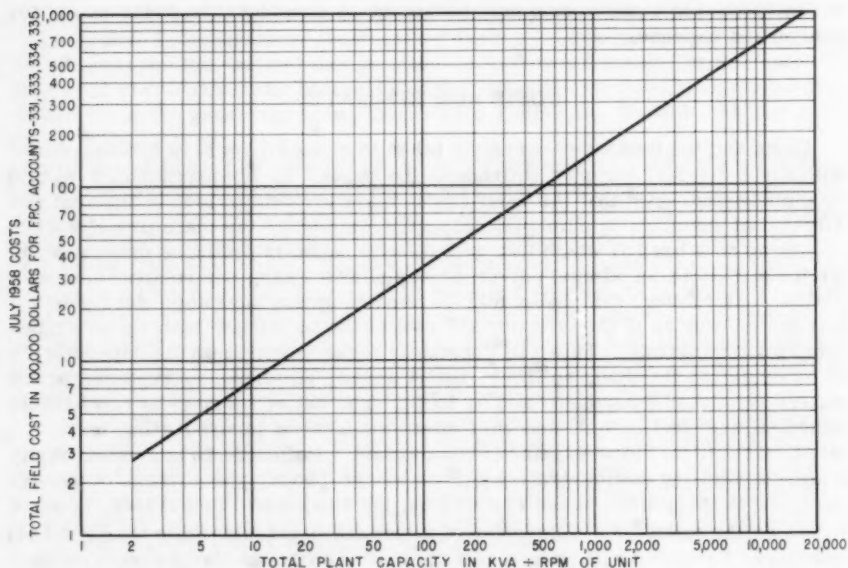

FIG. 9.—CURVES FOR USE IN MAKING RECONNAISSANCE ESTIMATES

and Table 2 show such a curve developed by the Estimates and Analyses Branch for use within the USBR. If desired, the plant may be satisfactorily estimated from the following relationships for an average installation:

FPC Account No. 331 (exclusive of the operators'
  residences) .... 30% to 35% of total

FPC Account No. 333 .... 55% to 50% of total

FPC Account No. 334 ....   10% of total

FPC Account No. 335 ....   5% of total
                                              ─────────────
                                              100% total

The preliminary or feasibility estimate is undertaken after the site is selected and the plant capacity determined. Quantity estimates of the various ar-

TABLE 2.—ESTIMATING DATA-FOR USE IN MAKING RECONNAISSANCE
ESTIMATES—HYDRO-ELECTRIC POWER PLANT COSTS

Total $\frac{KVA}{RPM}$		Percent of curve cost F. P. C. account number			
From	To	331	333	334	335
0	10	34.0	46.0	13.3	6.7
10	20	33.0	47.0	13.3	6.7
20	30	32.5	48.0	13.0	6.5
30	40	32.0	49.0	12.7	6.3
40	60	32.0	50.0	12.0	6.0
60	80	31.5	51.0	11.7	5.8
80	120	31.0	52.0	11.3	5.7
120	150	31.0	53.0	10.7	5.3
150	200	30.5	54.0	10.3	5.2
200	300	30.0	55.0	10.0	5.0
300	500	30.0	56.0	9.3	4.7
500	700	29.5	57.0	9.0	4.5
700	900	29.0	58.0	8.7	4.3
900	1,300	29.0	59.0	8.0	4.0
1,300	1,700	28.5	60.0	7.7	3.8
1,700	2,300	28.0	61.0	7.3	3.7
2,300	3,000	28.0	62.0	6.7	3.3
3,000	4,000	28.0	63.0	6.0	3.0
4,000	5,000	27.5	64.0	5.7	2.8
5,000	7,000	27.5	65.0	5.0	2.5
Over	7,000	27.0	66.0	4.7	2.3

Notes.—Costs represented by the curve in Fig. 9 are average total field costs for F.P.C.
Accounts 331, 333, 334, and 335, exclusive of costs for investigations, engi-
neering, and general expense.

Size, location, accessibility, foundation, and operating conditions should be an-
alyzed and the curve costs adjusted to cover the conditions peculiar to the
plant under consideration.

Costs of penstocks, gates, valves, trashracks, and other power facilities in and
near the dam must be added to the curve cost to obtain the total field cost of
the powerplant exclusive of land and rights, clearing, and relocations of ex-
isting property.

Structures and Improvements = F.P.C.   331
Waterwheels, Turbines, and Generators = F.P.C.   333
Accessory Electric Equipment = F.P.C.   334
Miscellaneous Powerplant Equipment = F.P.C.   335

To compute "RPM OF UNIT," first estimate the number and capacity of units
required. Do not use units larger than 125,000 kw. If more than one unit is
required, use an even number of equal-capacity units. Then,

$$RPM \text{ of Unit} = \frac{\sqrt{H \times 535}}{\sqrt{\frac{KW \text{ of unit}}{\sqrt{H}}}}$$

in which H = average head in feet

To compute "Total Plant Capacity in KVA," multiply total KW of units by 1.11
(0.9 power factor assumed).

rangements are prepared to aid in the selection of the scheme. These are prepared from small scale layouts (1/20 in. equals 1 ft) entailing the preparation of one drawing to show the particular arrangement of the powerplant. The civil-structural engineer preparing the estimate for Account No. 331 can readily accomplish his work by computing the following quantities:

1. Excavation
2. Backfill
3. Substructure concrete (through the roof of the draft tube)
4. Intermediate structure concrete (heavy walls, floors, and framing)
5. Second-stage concrete (turbine encasement and generator foundations)
6. Structural steel (columns, roof, and crane girders)
7. Superstructure enclosure walls (steel, block, and so forth)
8. Miscellaneous items (seals, finishes, miscellaneous metal)

Item 8 for an average surface plant with a tail water below the generator floor will approximate 25% to 30% of Items 3, 4, 5, and 6. The estimate for reinforcing steel and cement would be included in the unit price for the respective concrete items.

The engineer's estimate and the drawings and specifications data from which it is made reflect the considered studies and opinions of a particular organization. It is the way the project is to be built. Many minor alternatives and refinements are usually made between the preliminary estimate and the issuance of the construction specifications. These alternatives should be estimated comparatively, that is, without entailing all features of the project as is usually necessary during the reconnaissance or preliminary stages.

Today (1961), it is found that the proper and correct representation of the work to be performed under a specifications is more important than ever before. Ambiguities and changes invariably increase costs, and refinements introduced after the contract for the work has been awarded are usually lost in the completed job because of "changed conditions." The designer today is then faced with the dilemma of selecting his refinements and completing all desirable alternative studies in the time allotted for preparation of the specifications and the engineer's estimate. During this time one scheme must be chosen and sufficient detail given so that the contractors may bid intelligently and competitively. To provide as much time as possible for these alternative studies and to provide sufficient firm detail for sound competitive bidding, the Bureau currently uses two proposals that minimize the time required for the preparation of the specifications data for the powerplant structure. The first is to award two contracts, one for the skeleton first-stage construction (prime contract) and one at a later date for the installation of the equipment and the second-stage concrete (completion contract). Some of the variables inherent with the purchase of the equipment are thereby left out of the prime contract. The second measure is the preparation of a "structural arrangement" drawing or drawings. This is usually done to a scale of 1/16 in. equals 1 ft and shows all wall, floor, and column thicknesses so that the contractor may estimate his formwork. In addition all required construction joints are shown and optional construction joints (contractor's option) are indicated. Standard details such as roofing, seals, stairs, embedded metalwork, expansion and contraction joints, floor drains, keyways, blockouts, control joints, cover slabs, and so forth, have been prepared on letter-sized standard drawings that can be "pulled" for a particular job. The preparation of drawings and details required

for a firm bid are thereby minimized, and the maximum amount of time is utilized prior to the specifications issuance for studies and refinements.

The actual construction drawings (1/4 in. to 1 ft) showing the assembled details are prepared prior to the time required by the contractor's construction schedule. This has proved successful, and as one "structural arrangement" and a set of standard details often portrays the work included on 10 or 15 construction detail drawings, the time saved prior to specifications issuance is self-evident. These measures have reduced costs, because it is as important to present all proposed refinements in the specifications today as it is to conceive the refinement.

The preparation of the aforementioned estimates has a two-fold purpose. The first is to assist in the selection of the proper layout for a particular development. Second, is to establish the benefit-cost ratio of the plant. The latter is normally done by dividing the annual value to be received from the development by the annual costs. The annual value of the power is based on the value of the dependable capacity (kilowatts), the nominal prime energy (kilowatt-hours), and the secondary energy (kilowatt-hours). The annual costs are the sum of the fixed charges, operation and maintenance, and administrative and general expense. The fixed charges are usually a percentage of the capital cost, and the others are related to the installed capacity. Curves for estimating these percentages are available from an FPC publication[11] and from text books on powerplant design. Fringe benefits or costs, such as headwater benefit payments, must be included to establish the correct ratio. This benefit-cost ratio is computed and checked during all three estimate stages to confirm the efficiency of the plan and to support the selected capacity.

CONCLUSIONS

The current developments in the design and layout of hydro-electric powerplants have been presented herein. This is a challenging and exacting field of endeavor, and it is believed that the solutions offered will prove provocative to the engineers engaged in it. Should this paper than help to disseminate our knowledge and encourage further progress, it will have achieved its purpose.

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11 "Electric Utility Cost Units and Ratios," FPC publication S-18.

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UNDERGROUND POWER PLANTS IN YUGOSLAVIA

By V. M. Yevdjevich,[1] F. ASCE

SYNOPSIS

Yugoslavia has (as of 1961) twelve underground water-power plants in operation and three under construction, two of which exceed 400 mw. The majority are constructed in limestone, which has been found satisfactory for such plants. The plant chamber is generally near the face of the mountain, to reduce the length of access, cable, and tailrace galleries.

INTRODUCTION

The construction of underground water-power plants started in Yugoslavia before World War II, with the water-power station Vinodol in Croatia. Two underground water-power plants in Slovenia, Dablar (1939) and Plave (1940), were built before the war on the Socha River by the Societa Adriatica di Elettricita of Venice, Italy. These plants, based on the new border between Italy and Yugoslavia, are presently in Yugoslavia. Vinodol was put in operation in 1952, and between 1952 and 1957 nine other underground water-power plants were placed in operation. Some special types of underground water-power plants, such as the pit-type (station in a vertical large pit; Moste power plant), and the semi-underground power plant (one side of the underground chamber being the free surface of the nearly vertical rock side; Mesici power plant) will not be dealt with herein. However, some features of other underground

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water-power plants, either under construction in 1958-1960 or to be constructed in the near future, will be presented.

One reason that some of the most important water-power plants have been or are being constructed as underground plants is to allow for greater economy and wartime security.

DATA ON UNDERGROUND POWER PLANTS

The main data on underground water-power plants, in operation and under construction, are given in Table 1.

GEOLOGY OF UNDERGROUND POWER PLANTS

The geology of Yugoslavia has a decisive influence on the layout of the underground water-power plants. The geological history of many parts of

TABLE 1.—UNDERGROUND WATER-POWER PLANTS IN YUGOSLAVIA

Name of Plant (1)	Turbines, Number and Type[a] (2)	Plant Capacity, in Mw (3)	Head, in Meters (4)	Type of Rock (5)	Year of Initial Operation (6)
Doblar	3 Fv	33	49	Limestone	1939
Dubrovnik	4 Fv	416	272	Limestone	Under construction
Glava Zete	2 Kv	5	21	Limestone	1955
Jablanica	6 Fv	168	105	Werfen schists	1955
Jajce I	2 Fv	50	91	Limestone	1957
Jajce II	3 Fv	27	53	Limestone	1955
Mavrovo	4 Pv	150	550	Quartz schists	1957
Medjuvrsje	2 Kv	7	22	Diabase-horn-blende	1955
Ovcar-Banja	2 Kv	7	20	Limestone	1955
Plave	2 Kv	17	25	Limestone	1940
Ras	2 Fv	7	163	Limestone	1955
Senj	4 Fv	240	428	Limestone	Under construction
Split	4 Fv	425	269	Limestone	Under construction
Vinodol	3 Ph	84	660	Limestone	1952
Vrla I	4 Pv	48	330	Quartz-diorite	1955

a The symbols have the following meanings:
F = Francis; P = Pelton; K = Kaplan; v = vertical shaft; h = horizontal shaft.

Yugoslavia has been characterized by rather heavy movements through the ages.

The igneous rocks in Yugoslavia, such as granite, are not considered to be the most suitable for underground caverns of large span, as is sometimes the case (for example, in Sweden). The mountain-building forces have changed these rocks in the majority of cases, so that they are full of numerous fractures, and are weathered to a great depth. They very often creep, when the excavations are not carefully protected. Many faults occur, frequently with bad rock on one or on both sides of the fault.

Sedimentary rocks, however, such as limestone and dolomite are good for construction of underground power plants. Nine of the twelve underground

plants are built in limestone, and only three of the plants are in other rocks (Table 1). The largest power plant under construction, Split in Dalmatia (425 mw), and two other large power plants (Dubrovnik 416 mw, Senj 240 mw) are being built with underground power chambers in limestone. The limestones are of every age, but mostly triassic, jurassic, or cretaceous.

This high proportion (11 out of 15) of the underground water-power plants being built in limestone is due to the following facts: (a) limestone is the common rock in Yugoslavia (covering one-third of the mountainous area of the country); (b) experience shows that this rock is the most suitable for underground power plants because of the optimum hardness; (c) it appears very often in massive and unfracturated blocks; and (d) if the rock has initial fractures, they are normally well recemented afterwards. The limestone in Yugoslavia is mostly karstified, with the well-known karstic phenomenon, which is developed in the most complete and typical forms. Therefore, it is very pervious rock.

Limestone is such a hard rock that a smaller excavation volume and concrete lining for a given type of plant are necessary. On the other hand, the rock is not so hard as to make excavation too expensive. The limestone in Yugoslavia, when in good massive blocks and well recemented, could be considered as having almost the optimum quality of rock for underground work. It is possible, in most cases, to construct vertical side-walls with a light concrete arch protection for the cavern (Figs. 1(b), 1(d), 2, 5(b)).

The permeability of the limestone, which in the underground excavation results in heavy pumping of seepage water, also presents a potential danger of underground floods and accidents during excavation. For this reason, the so-called "Swedish" type of underground water-power plant, with the caverns straight below the intake structure and with a long tailrace tunnel, has been abandoned wherever it has been studied. The "upper derivation tunnels" have always been considered more suitable for underground work in limestone.

The four underground power plants built in other types of rock are: (1) Jablanica (168 mw) constructed in werfen schists, which were unusually thick and in very good condition for easy and safe underground work; (2) Mavrovo (150 mw), constructed in hard quartz schists, which were very resistant for underground excavation; (3) Vrla I (50 mw), in quartz-diorite intrusions in paleozoic schists, where the contacts were very troublesome during the excavation; and (4) Medjuvrsje (7 mw) in diabase-hornblende, having a very unstable quality of rock.

The potential creep of the rock was eliminated by the use of a horseshoe-shaped profile, with rather heavy concrete lining (Figs. 3(c), 6(b), and 8(b)).

In the cases of the Mavrovo and Medjuvrsje underground plants, the problem of underground water had to be solved. In the case of Medjuvrsje, this water threatened to disintegrate the concrete. A storage reservoir close to the cavern tended to increase the already existing water table of mountain water around the cavern. After the reservoir was filled, water began to flow, causing heavy and rapid disintegration of concrete. A special net of drainage holes and drainage pipes was installed to evacuate the underground water before it came in contact with the concrete lining of the cavern.

SOME GENERAL PROBLEMS TREATED IN THE LAY-OUT OF UNDERGROUND POWER PLANTS

In order to avoid great spans in the underground caverns, the trend was to design and construct longer but narrower caverns. For that reason, in some
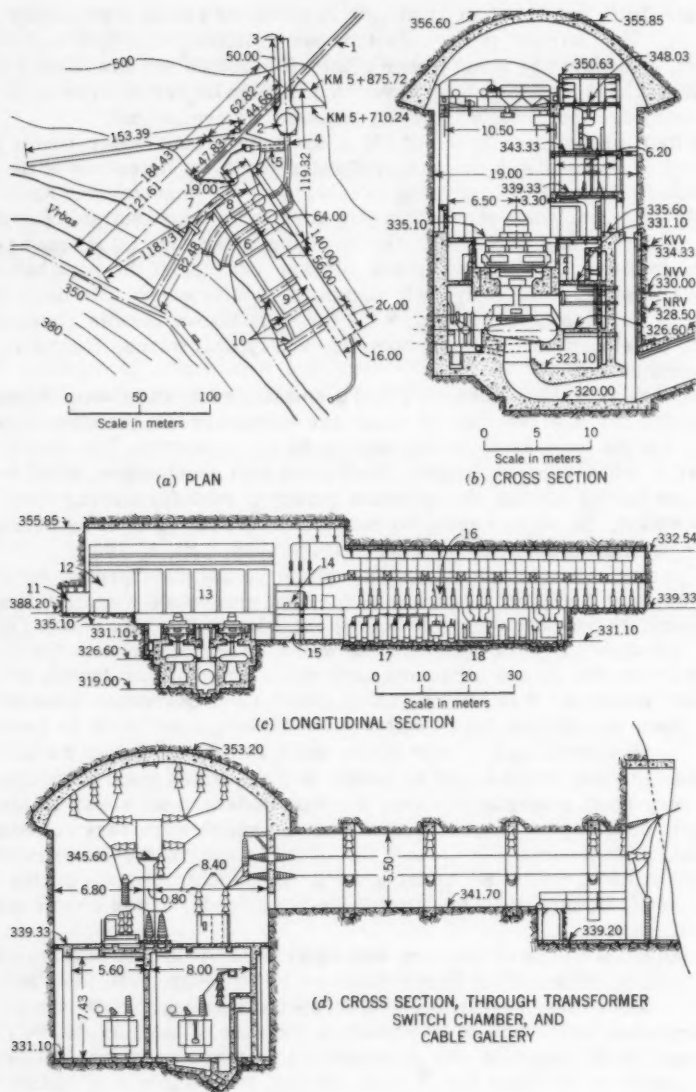
(a) PLAN

(b) CROSS SECTION

(c) LONGITUDINAL SECTION

(d) CROSS SECTION, THROUGH TRANSFORMER
SWITCH CHAMBER, AND
CABLE GALLERY

Scale in meters

FIG. 1.—JAYCE I POWER PLANT: $P_i = 48$ mw; $Q_i = 60.8$ cu m per sec; $H_{max} = 98.25$ m; $E = 220 \times 10^6$ kwh; two vertical Francis turbines; pressure tunnel length = 5,713 m and diameter = 5.40 m: (1) pressure tunnel; (2) surge tank; (3) upper chamber of surge tank; (4) penstock; (5) galleries; (6) main underground cavern; (7) access gallery; (8) tailrace canals; (9) transformer and switch chamber; (10) 110 kv free outlet; (11) workshop; (12) erection space; (13) main cavern; (14) control room for high voltage; (15) control for low voltage; (16) switch for 110 kv; (17) 10.5/110 kv transformers; and (18) 110/35 kv transformers.
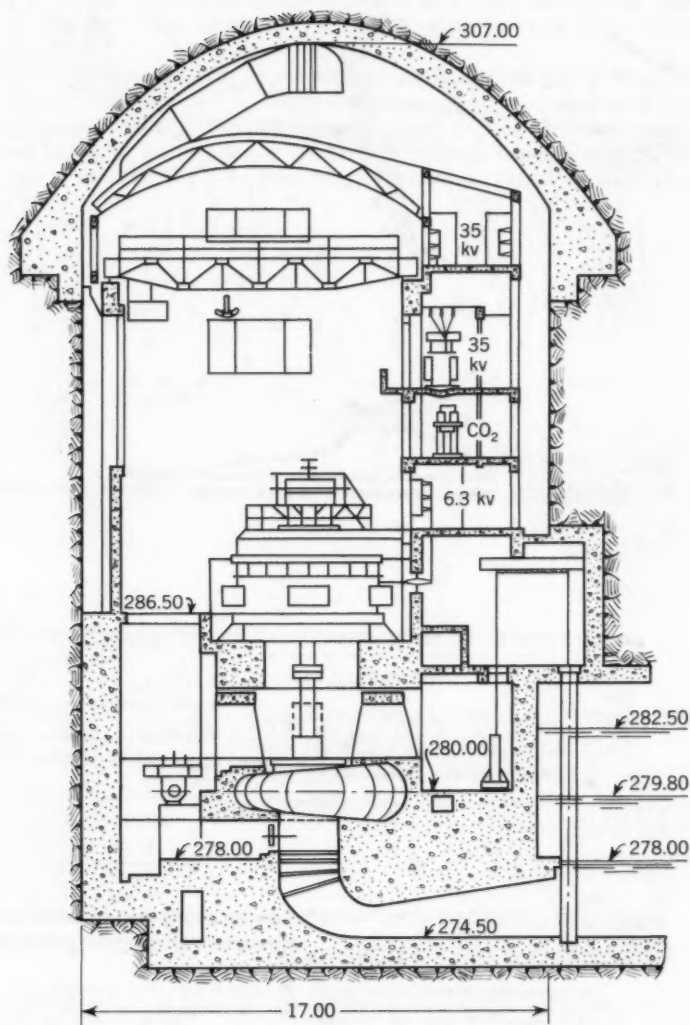
FIG. 2.—JAYCE II UNDERGROUND PLANT: P_i = 30 mw; Q_i = 80 cu m per sec; H = 53 m; E = 180 X 10^6 kwh; diversion tunnel length = 2,804 m and diameter = 5.50 m; and four vertical Francis turbines, with free tailrace canal, 146m.
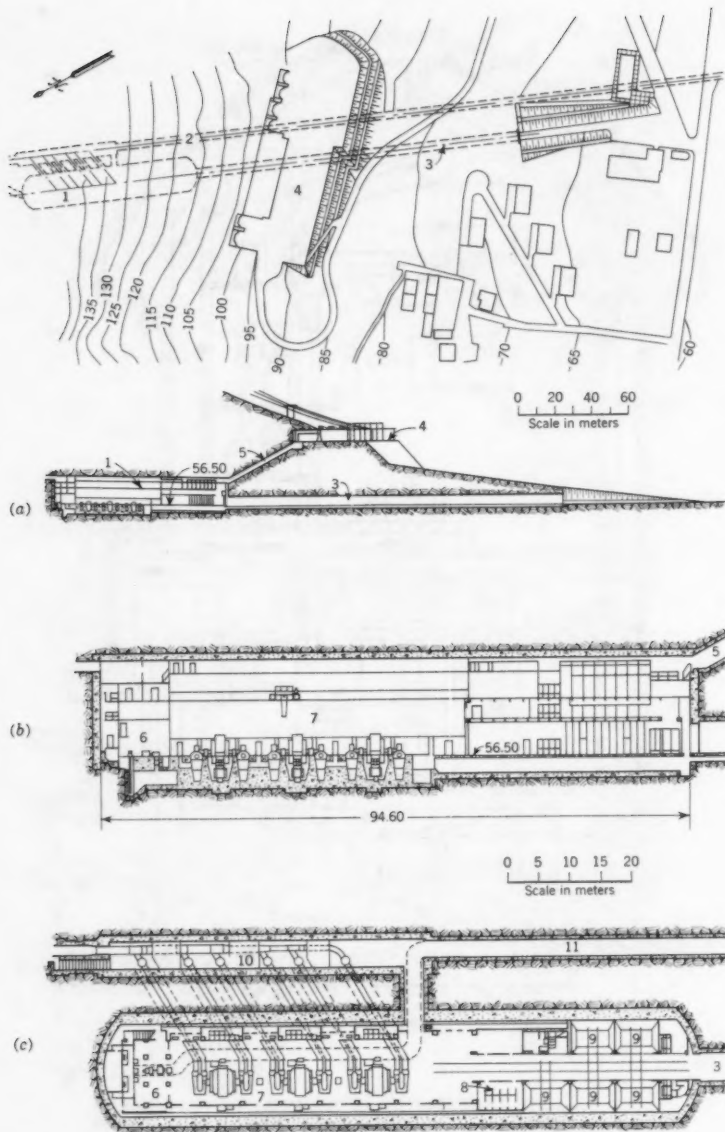
FIG. 3.—VINODOL UNDERGROUND PLANT: $P_i = 84$ mw; $Q_i = 15$ cu m per sec; $H = 645.5$ m; $E_o = 180 \times 10^6$ kwh; and three units each of two Pelton turbines with horizontal shaft: (1) Power plant cavern; (2) tailrace gallery; (3) access gallery; (4) switch yard and transformers to 110 kv; (5) cable gallery; (6) house turbine; (7) three units, each with two Pelton turbines; (8) control; (9) transformers; (10) valve and water distribution chamber; and (11) tailrace canal.

cases, a special underground valve cavern was built, so that the main underground chamber was constructed with a shorter span (Figs. 3(b), 3(c), 6, 8(a), and 8(b)).

The free laid penstocks, in the sloping galleries and in the bifurcation, are considered, in many cases, to be a potential source of danger of flood in the power caverns. To protect the power plant against the consequences of an eventual break of the penstocks or other equipment, either a special evacuating emergency gallery (Vrla I, Fig. 8(a); Jablanica, Fig. 6(a)) was needed, or a

(a)

(b) CROSS SECTION   (c) LONGITUDINAL SECTION
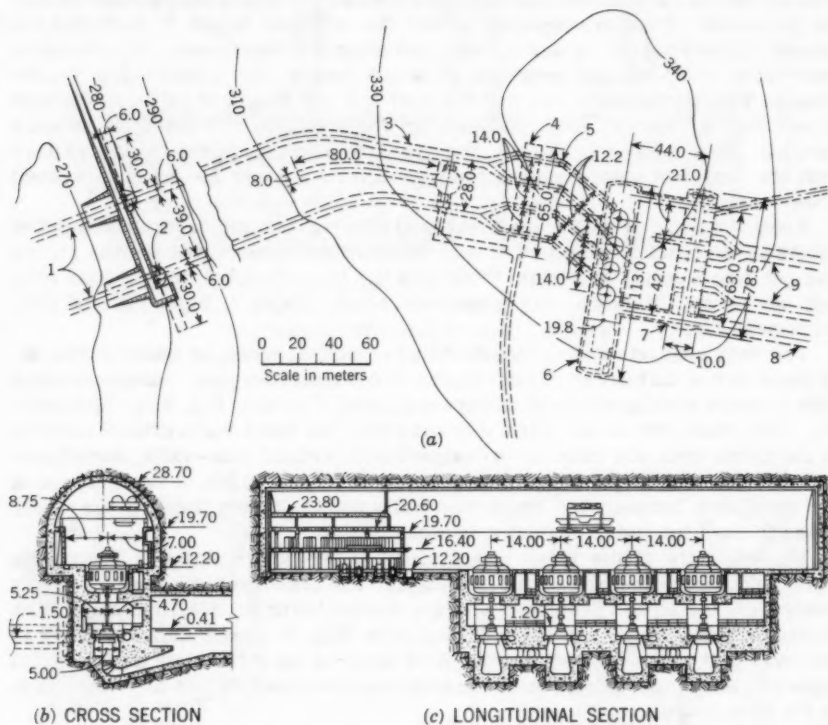
FIG. 4.—SPLIT POWER PLANT: P_i = 425 mw, first stage: One tunnel and 212.5 mw; Q_i = 220 cu m per sec; H = 269 m; E = 1,960 X 10^6 kwh; four vertical Francis turbines; two tunnels, each 9,800 m long and diameter = 6.00 m: (1) Headrace tunnels; (2) surge tank No. I; (3) surge tank No. II; (4) upper valve chamber; (5) penstocks; (6) underground cavern; (7) transformer chamber; (8) access; and (9) tailrace tunnels.

connection of the penstock galleries or underground valve chambers with the tailrace canal (Mavrovo, Fig. 7; Vinodol, Figs. 3(a) and 3(b)). A special safety steel gate was mounted between the underground power cavern and the penstock gallery or the underground valve chamber.

The main transformers were variously located: (a) inside the main underground cavern (either in an extension of it, as at Jajce I, Figs. 1(c) and 1(d); or opposite the generators, as in Jablanica, Fig. 6(b)); (b) in separate caverns (Split, Fig. 4(a); Dubrovnik, Fig. 5; Mavrovo, Fig. 7); or (c) outside, above ground, in a protected place, such as at the foot of a hill (Vrla I, Figs. 8(a) and 8(b); Vinodol, Fig. 3(a)). The separate cavern location, for main transformers, seems to be preferred in the newest design trends.

The general tendency in designing is directed toward locating the underground power cavern close to the free surface, at a depth that gives enough stability for the structures and that, at the same time, is sufficient for security purposes. This arrangement gives the shortest length of galleries for access, cable conduits, water outlets, and emergency purposes. Experience in excavating steep sloping penstock galleries (under the construction conditions in Yugoslavia) indicates that the cost of a unit length of gallery is lowest either for a vertical shaft gallery or for a gallery of mild slope. When a vertical shaft was not adopted, the trend was to design the sloping gallery with the smallest slope, assuring just enough rock cover for the underground power cavern.

From the underground transformers (110 kv) in the main caverns, the outlet cables to the outside switch yard were designed and constructed as free cables and put into special galleries. This was the case when building in good rock and where the distance was relatively short. (Jajce I, Figs. 1(a) and 1(d); Jablanica, Fig. 6).

The rock characteristics (possibilities of sildes, creep, or water seepage), in many cases, influenced the designers to construct the power-house caverns with protection walls of a special shape and lining (Vinodol, Fig. 3(c); Jablanica, Fig. 6(b); Mavrovo; Vrla I, Figs. 8(b) and 8(c)). The large underground caverns in limestone rock are most often designed with vertical side-walls, sometimes without concrete lining (Figs. 1(b), 1(c), 1(d), 2, and 5(b)). A small space of air insulation between the protection walls and the inside lining of the power house is used for water drainage, in many cases.

The majority of the large underground water-power plants in Yugoslavia were built or are being built in two stages. The underground power cavern is always excavated and protected with a concrete lining for the final stage. The Mavrovo (Fig. 7), Vrla I (Fig. 8), and Split (Fig. 4) power plants are built in this way and are equipped for the first stage of operation with half the final capacity. All of the underground excavation in the power cavern has been made in the first stage.

Because the underground power caverns are located near the free-rock surface, the tailrace tunnels are, in all cases, designed and constructed to be used with free-surface flow, and with a high gallery or canal, to assure the operation of the plant during high floods as free-surface flow.

The close location of the power chamber to the valve chamber sometimes raises the problem of the stability of the dividing rock mass, as in the case of Vinodol, (Figs. 3(b) and 3(c)), but in most cases (Vrla I, Figs. 8(a) and 8(c); Jablanica, Fig. 6) the distance between them could be decreased.
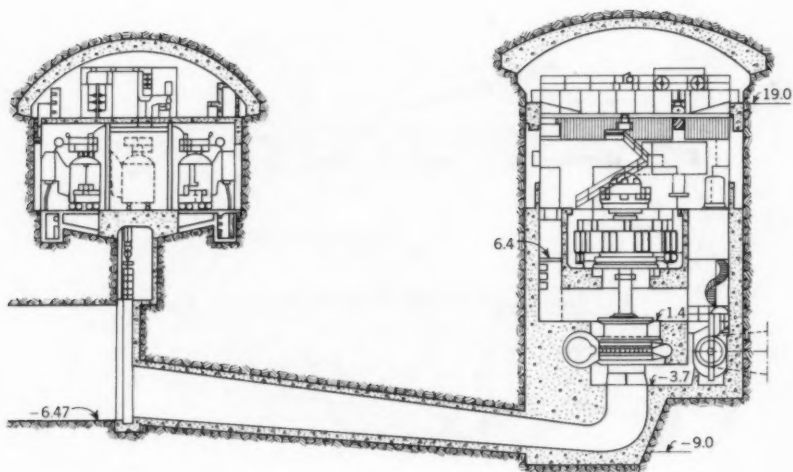

EFFORTS TOWARD FURTHER IMPROVEMENT OF DESIGN

The design of underground power plants is directed toward (a) the proper selection, from a geological point of view, of the site for the underground

(a) MAIN UNDERGROUND POWER CAVERN
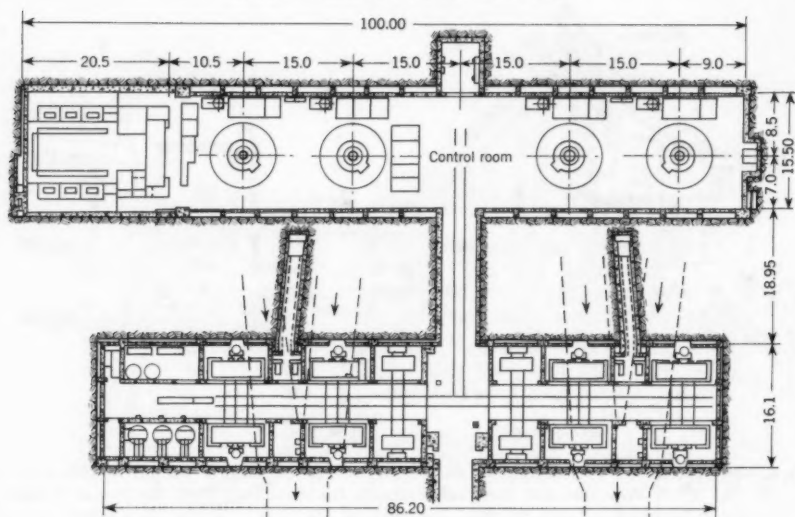
(b) TRANSFORMER UNDERGROUND CHAMBER

(c) CAVERN FOR UNITS

(d) TRANSFORMER CAVERN

FIG. 5.—DUBROVNIK PLANT: P_i = 416 mw; Q_i = 180 cu m per sec; H = 272 m; E = 2,010 X 10^6 kwh; and four vertical Francis turbines.
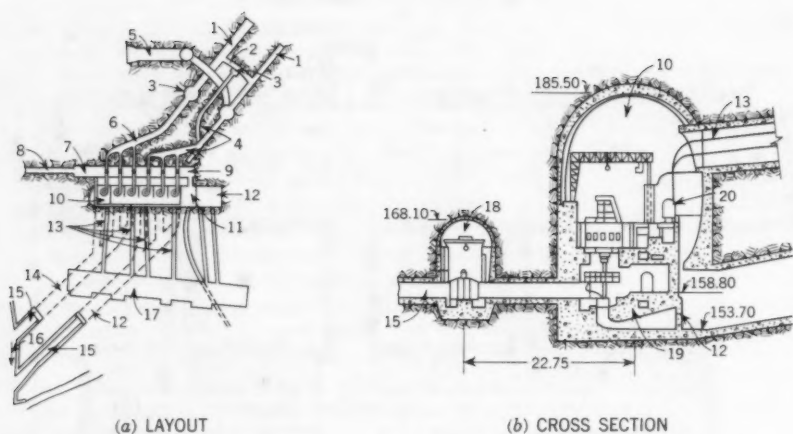
(a) LAYOUT   (b) CROSS SECTION

FIG. 6.—JABLANICA PLANT: P_i = 168 mw; Q_i = 180 cu m per sec; H = 105 m; E =
750 X 10^6 kwh; and six vertical Francis turbines: (1) Two headrace tunnels;
(2) connection gallery between two tunnels; (3) control valve; (4) access; (5) two
surge tanks, working separately or jointly; (6) penstocks; (7) valve cavern; (8)
gallery for emergency drainage of water; (9) valve chamber; (10) power cavern;
(11) access; (12) inside switch for 35 kv; (13) galleries for the 110 kv cables;
(14) tailrace galleries; (15) canals; (16) Neretva River, switch yard for 110 kv;
(17) penstock; (18) valve cavern; (19) vertical Francis turbine; and (20) 110 kv
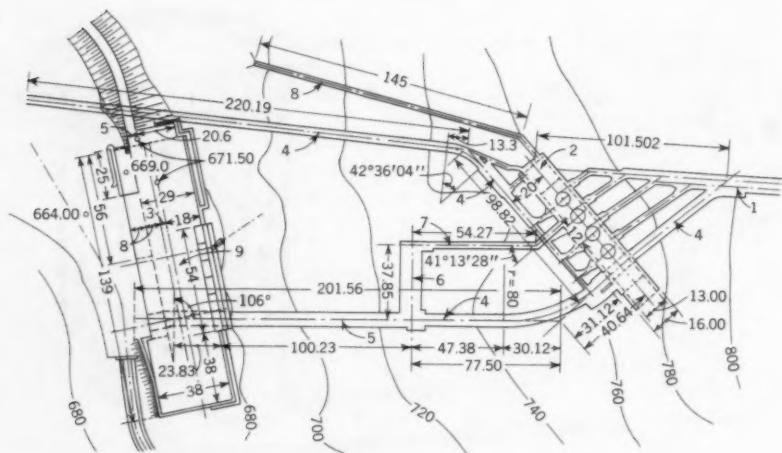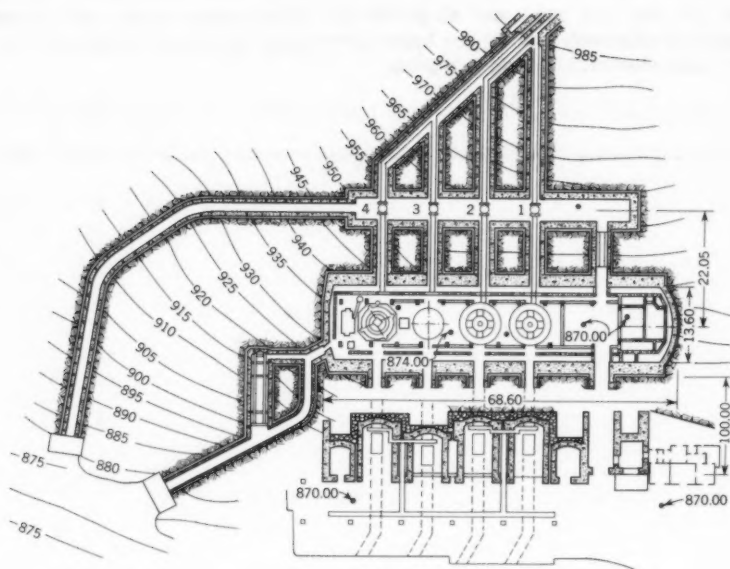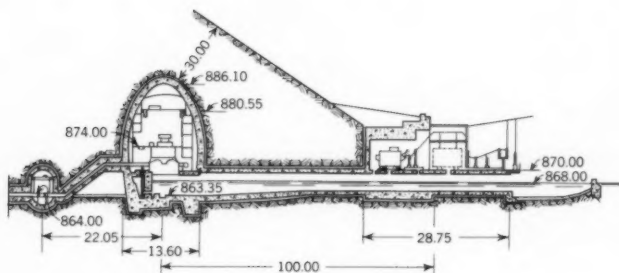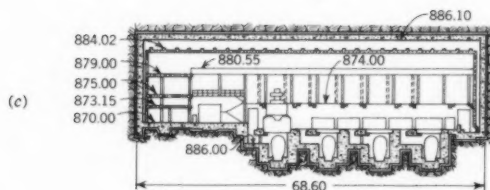transformers.

FIG. 7.—MAVROVO PLANT: P_i = 150 mw; Q_i = 32 cu m per sec; H = 550 m; E = 300
X 10^6 kwh; and four Pelton turbines with vertical shafts; (1) Two free penstocks
in the gallery; (2) power cavern with four units; (3) water outlets; (4) tailrace
canal; (5) access galleries; (6) 110 kv transformer cavern; (7) cable gallery;
(8) emergency exit; and (9) switchyard.

FIG. 8.—VRLA I PLANT: P_i = 48 mw; Q_i = 16 cu m per sec; H = 330 m; E = 50 X 10^6 kwh; four Pelton turbines with vertical shaft; and the valve chamber is low because it was constructed for the previously planned vertical Francis turbines instead of Pelton turbines actually built.

cavern; (b) a more detailed study of the economic location of the cavern, considering a minimum cost for all galleries leading to or from the cavern; (c) the study of the best size and shape of the underground cavern with respect to equipment selected; and (d) the better functional lay-out of access, water conduits, and electrical conductor runs.

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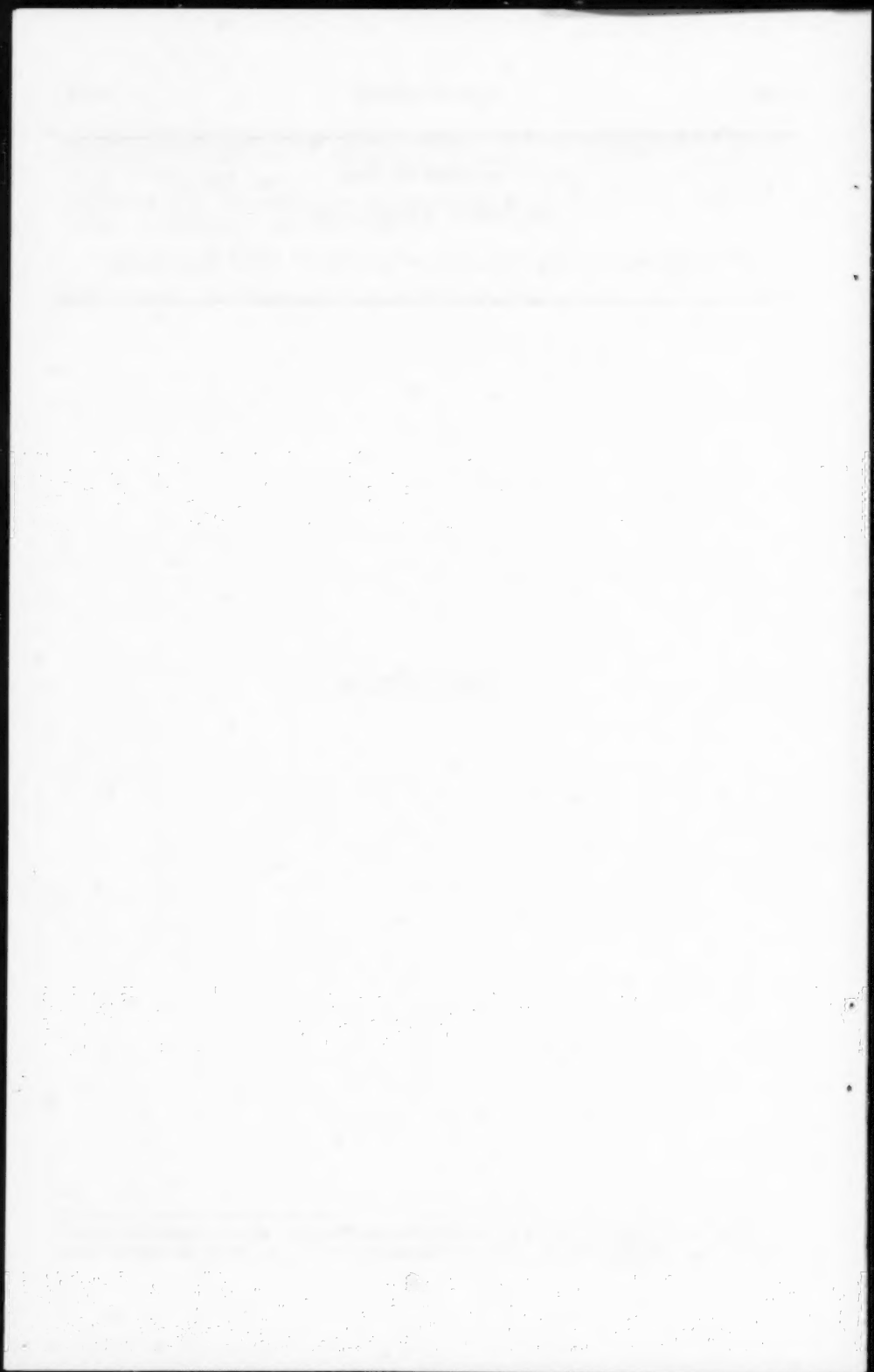
Proceedings of the American Society of Civil Engineers

DISCUSSION

DIGITAL COMPUTERS FOR TRIAL-LOAD ANALYSIS OF ARCH DAMS[a]

Closure by L. R. Scrivner

L. R. SCHRIVNER.[12]—The early phases of the programming to produce data for the trial-load method of stress analysis for arch dams are described. Additional programming has since been done which has removed some of the restrictions listed under the heading "Summary and Conclusions" in the paper. The following numbered changes refer to the restricitions as previously listed:

1. Upstream (extrados) radius may change at the plane of center;

2. abutment angle may be any angle between 10° and 90°, and could be carred to six decimal places if desired;

3. contact of concrete to rock may be radial, or a triangular wedge of concrete may be included in the analysis of arches excavated to half-radial lines; and

4. approximately two-thirds of the programming required to permit loads to vary linearly between cantilever points has now been done. Deflections would then be computed at points of intersections of the cantilever with the arches.

Wengler inquired about the need for choosing smaller incremental angles and therefore more arch points. On completion of current programs, the writer will be able to run a simple check to determine how much the results are affected by the number of points. The capacity of the computer of the USBR limits the current program to six incremental angles per arch. One to six incremental angles may be used and Table 6 indicates the time required in the computation of tables of f(x) for four equal angles and varying number of unequal angles.

Time required for complete data is approximately 30 sec for each different angle for which f(x) is required. The time is reduced approximately 30 sec for any load omitted. A six-part arch with three triangular loads would, therefore, require about 645-90 or 555 sec to compute f(x), compared to the 125 sec for an arch with four equal angles. The big advantage will be that arch and cantilever deflections will be computed at the same points and may then be compared directly instead of graphically, as at present (1961).

No decision has been reached as to the best means for computer solution after unit deflections have been found.

As suggested by Sarkaria, it will be possible to develop data to be used as a guide as far as minimum structures are concerned. The problem of what constitutes a minimum structure must be solved first.

Other problems can be much more readily investigated by the computer, for example, effects on the stability of an arch due to less than radial excavation. A similar study is being undertaken on a current job. Effects of overhand could also be similarly studied.

TABLE 6.—TIME TO COMPUTE $f(x)$[a]

Angles (1)	Angles used (2)	Type of Analysis		
		Crown (3)	Radial (4)	Complete (5)
4 equal angles	4	35	65	125
2 unequal angles	3	25	50	90
3 unequal angles	6	40	85	165
4 unequal angles	10	70	145	270
5 unequal angles	15	120	235	465
6 unequal angles	21	170	350	645

[a] Time in seconds—Loads linear between points.

At some future time a series of cures such as suggested by Sarkaria will undoubtedly be developed. A major problem is the large number of variables that may be included, such as:

1. T/r, thickness to average radius;
2. E_r/E_C, ratio of rock to concrete modulii;
3. T_a/T_0, thickness at abutment to thickness at crown;
4. arch central angle; and
5. other factors which influence abutment constants.

The discussions by Wengler and Sarkaria were constructive and helped to bring into focus many of the problems which are yet to be solved.

DESIGN OF ARCH DAMS BY TRIAL-LOAD METHODS OF ANALYSIS[a]

Closure by Merlin D. Copen

MERLIN D. COPEN,[18] F. ASCE.—Adolf A. Meyer, F. ASCE, refers to the writer's statement regarding the practicability of dynamic loads on structural models. Since the publication of this paper, the writer has had the opportunity to visit structural model laboratories in Lisbon, Bergamo, and Paris. During the time that dynamic model tests have been attempted, the only laboratory in which they have been performed with any success is the Instituto Sperimentale Modelli e Strutture in Bergamo, Italy. But even here the success has been limited to an approximation of the intensity of an earthquake which will produce failure in the structure. Some of the problems faced by this laboratory and not yet completely solved include: similitude conditions, application of dead loads, measurement of strains during vibration, inability to apply vibration in more than one direction simultaneously, and restriction to a mathematically defined rather than random-type vibration. The complete and effective application of dynamic loads to a structural model of a dam is still not an accomplished fact.

The direction in which an earthquake is applied is no problem analytically. Earthquake acceleration in both a vertical direction and parallel with the arch chord have been applied and analyzed by trial-load methods. Neither of these conditions have been found to be critical in dams designed by the USBR. It should be remembered that the dynamic effect of waterload is the principal force induced by earthquake, and this is usually a minimum in the direction of the arch chord. Only if a dam is thin, has a large central angle, and is in a wide valley would an earthquake acting parallel with the arch chord be in the critical direction. Studies in the USBR indicate that vertical earthquake acceleration has a negligible influence on arch dams, resulting primarily in a minor redistribution of load from the vertical to the horizontal direction, with only small stress changes.

The writer is not aware that double curvature type arch dams have shown superior performance in withstanding seismic loads. The shape of dam should be determined primarily by site conditions, that is, canyon shape, hydrological factors, foundation properties, height of structure, etc. Models of double curvature arch dams which indicate unusual performance, do so because they are most satisfactory for the particular site in which they are constructed.

The manner in which dead loads are carried by an arch dam is primarily dependent on the method of construction. If the dam is built to its complete height without closing its construction joints, the dead load is carried by cantilever action. If, however, the joints are grouted progressively during the construction period, some of the dead load is carried in arch action. The ef-

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a August 1960, by Merlin D. Copen (Proc. Paper 2569).

18 Engr., Bur. of Reclam., U. S. Dept. of the Interior, Denver, Colo.

fects of construction and grouting procedures are included in the analyses of the arch dams by the trial-load method.

The writer is acquainted with the "plunging" arch theory, and agrees that resultant forces are transferred through an arch dam along inclined surfaces. The direction of such lines and the magnitude of stress in that, or any other direction, can easily be obtained from a complete trial-load study. Unit elements may be chosen in any direction the designer desires. If all components of stress, both direct and shearing, are computed for these elements, the designer may then determine stresses in any direction required. Horizontal and vertical elements are used only because of simplicity. When all stresses are computed for these elements, lines of principal stress or stresses in any direction can readily be obtained.

The writer is not acquainted with any type of concrete arch dam which cannot be analyzed by trial-load methods. The stresses and movements in double curvature, poly-centered and parabolic arch dams, as well as the simpler types, may be determined by this means.

It is most unfortunate that more comparisons between prototype behavior and predictions by trial-load analyses are not available. The necessity for economy in USBR operations requires that a minimum number of studies be made. Designs are generally based, therefore, on the most severe loading condition expected to ever exist. Since this situation is rarely, if ever, realized, it is seldom possible to make a comparison. Those comparative studies which have been completed indicate good agreement, and it is hoped that more of them can be made in the future.

The writer is not familiar with the publication mentioned by Meyer regarding a comparison of model and prototype measurements for ten dams made by Societa Adriatica di Elettricita, and would be most grateful for this reference.

J. A. Veltrop, M. ASCE, refers to the writer's statement regarding simplified analyses. These analyses are crown cantilever and radial deflection studies, and are most effective as a means of estimating final stresses from a complete analyses. This is made possible by numerous comparative studies of various shapes and sizes of arch dams for both simplified and complete trial-load studies.

Veltrop is correct in his statement regarding the location of arch and cantilever elements. Two other considerations are also of value. First, arches and cantilevers should be located in areas where high or unusual stress conditions are anticipated. Second, wherever possible arch elements should have a cantilever abutting at the same point, in order to properly evaluate the movements and stresses at the rock contact plane.

Concern is indicated by Veltrop regarding the sufficiency of three adjustments. The writer would first like to state that the shape of the dam has no relationship to this question. Complex shapes may complicate the computation of the properties and unit or initial movements of the elements, but after these factors are known, a complete solution is unaffected by shape. Likewise, Poisson's ratio effects should have no relationship with the number of adjustments required since this factor may be treated in a manner similar to an applied load. The number of adjustments needed depends of the desires of the engineer handling the job. As many as six may be used but as few as three are actually required. To reduce the number of adjustments necessitates a combination of deformations in such a way that one adjustment will achieve agreement of deformation in two or more directions. For example, movement in a

vertical direction produces slope in the horizontal elements. This slope results in a rotation about the radius which in turn produces tangential movement. By carefully analyzing and computing these movements, the vertical adjustment may be combined with the tangential adjustment. Using a similar procedure, it is possible to combine the three linear movements and three rotations into three adjustments: radial, tangential, and twist.

The effects of temperature change are included in both arch and cantilever elements. The effects of thrust on rotation and bending on rib shortening are also included in the trial-load analysis.

By using an electronic computer, the time required for a radial deflection analysis is not much more than that for a crown cantilever study. There would be little gained, therefore, in modifying the crown cantilever study with the effects of other than uniform loads.

In a crown cantilever analysis the dead load is usually assumed to be carried by the cantilever element. If the construction joints are grouted progressively, however, the effects of dead load may be divided between arch and cantilever elements.

The maximum allowable stresses permitted in arch dams built by the USBR are based on the results of a complete trial-load analysis. In assessing the adequacy of a design from the results of a crown cantilever or radial deflection adjustment, the judgment of the designer must be used to estimate the effects of tangential shear and twist. The differences in the maximum stresses, shown for such analyses in the paper, reflect the effect of canyon shape and arch properties on the tangential shear and twist. The maximum compressive stress permitted in an arch dam designed by the Bureau of Reclamation is 1,000 psi for normal full reservoir and earthquake load. (This assumes a concrete strength in 1 yr which is four times the maximum computed compressive stress.) For the same loading condition, tensile stresses should not exceed 100 psi. Maximum tensile stresses are sometimes permitted to exceed this limit if they occur only in limited areas or in places where little damage could result from cracking, such as on the downstream face of the dam.

The crown cantilever analysis is considered a minimum for obtaining stress estimates for preliminary designs. Wherever possible, a radial deflection analysis should be made since only a small amount of additional time is necessary and a better estimate of the load distribution is obtained. The time required to prepare the data necessary for an average radial deflection adjustment on an IBM 650 is approximately 2 hr. The actual adjustment requires from 8 hr to 16 hr, as opposed to 4 hr to 8 hr for a crown cantilever adjustment. The adjusting procedure is the most time-consuming part of the analysis if an electronic computer is utilized to obtain the preliminary data.

It is expected that ultimately the entire trial-load analysis will be handled in one operation on an electronic computer. The immediate plan is, however, to perform the trial process on the computer and prepare the intermediate material with desk computers.

The crown cantilever analysis is used in the preliminary stage of the design of a project. A radial deflection study is prepared for feasibility estimates, and a complete analysis for final design. In all these studies the most critical loading expected to ever occur is used. This generally includes temperature change, normal full reservoir, and earthquake.

The radial deflection adjustment merely provides an indication of the change in load distribution between the crown cantilever and the arch abutments. It

has little effect on the stresses at the crown section. The influence of twisting moments and tangential shears is much greater than the change found from a radial deflection adjustment.

Sarkaria comments regarding the influence of the elastic properties of abutment rock on the deflections and stresses in the dam. This is an area in which there is much information yet to be determined. Efforts are being made both in the United States and Europe to secure more data regarding the effect of abutment rock on the dam. In general, the writer has found that the behavior of the abutment has considerable influence on the deflections and stresses. A low effective modulus of elasticity (less than 1,000,000 psi) results in large movements of the dam, a tendency to equalize the stresses across the abutment section of the arches, reducing both tensile and compressive stresses, and an increase in tensile stresses in the area of the crown. A high effective modulus of elasticity tends, on the other hand, to increase tensile stresses at the abutment extrados and cause only small movements in the dam.

The agreement obtained between measured stresses in the prototype and analytical computations depends on a number of factors. Possibly the most important of these are:

1. The completeness of the computations—anything less than a complete trial-load analysis is inadequate;

2. the accuracy of the basic assumptions used in the analysis;

3. use of the same loading conditions in the analysis which are found on the prototype;

4. the nature and accuracy of the measuring instruments and the care with which they are placed in the dam;

5. the care with which readings are made and the accuracy of readings taken from instruments in the dam; and

6. the proper interpretation of strain measurements and their reduction to stresses in the prototype.

The writer believes that where these conditions are met, a good comparison is possible, and, indeed has been obtained, both in USBR studies and other parts of the world where similar comparative studies have been made. Quite often, it becomes difficult or impossible to meet the required conditions for good accuracy in either or both the computations or measurements. As a result, conclusions are sometimes arrived at without justification regarding the adequacy of either or both methods.

Regarding Sarkaria's suggestion for limits within which a particular type of analysis is sufficient, the writer would like to make these observations:

1. The crown cantilever analysis is not sufficient as a final means of stress determination for any dam, no matter how small.

2. The writer considers a radial deflection analysis to be a minimum requirement, and this only for small dams, say less than 150 ft high, and site conditions approaching an ideal situation.

3. A complete analysis should be made if a dam is higher than 150 ft or where difficult or unusual site conditions exist.

A temperature rise is included in the designs cited herein only because of the climatic conditions at the sites and the use of precooling techniques to reduce the temperature as low as economically and physically possible before grouting the joints. The temperatures used in stress studies are almost al-

ways the minimum expected to occur in the dam, and these are often drops as well as rises. The writer considers precooling to be an effective prestressing device for arch dams. Even if artificial cooling is not practical, if care is taken to choose a time when the temperature of the concrete is low, grouting can be accomplished to produce much the same effect as with precooled concrete.

Uplift has been included in trial-load studies but is now generally not used. The grouting and drainage procedures, of both dam and foundation used in the USBR, have been shown by measurements to greatly reduce the uplift. But even when assumed to exist, uplift results in only minor stress changes in the dam. If cracking occurs, however, the effect of uplift becomes important and is always included in the analysis. Either or both arches and cantilevers may be assumed to crack if tensile stresses become excessive, and full uplift would be effective over the entire cracked surface. A much better solution to this problem of excessive tensile stress, in the writer's opinion, is to design the dam in such a way that this condition will be avoided. In most dams, it is possible to accomplish this by careful study and the application of sound design procedures.

It should be noted by Sarkaria that the dams mentioned on the Salt River in Arizona were not designed but only analyzed by the USBR.

Several factors are important when considering the allowable magnitude of tensile stresses or means of measuring them or their effect:

1. Good concrete has a tensile strength of approximately 300 psi to 1,000 psi;

2. this strength increases where a lateral pressure is present, as in the case of a dam;

3. if cracking does occur, an arch dam is capable of readjusting itself readily to accommodate this situation; and

4. measuring devices placed on or near the surface are susceptible to external and skin conditions that are not representative of the whole structure.

In the case of Dam A, mentioned by Sarkaria, a crack is not expected to develop because:

1. The extremely severe load for which the dam has been analyzed will probably never be experienced;

2. the effect of creep in the concrete will tend to reduce this tensile stress; and

3. the strength of the concrete is great enough to prevent cracking.

DESIGN OF KARADJ HYDROELECTRIC PROJECT[a]

Discussion by Robert Alexander

ROBERT ALEXANDER.[3]—The authors state, "The Karadj Dam . . . is the culmination of many years of effort." The dam was considered "as early as 1937," and received further consideration after World War II, when increased growth of Tehran led to a critical situation in water supply and electrical energy.

Promotion of the project, which led to the Karadj Dam construction, dates from 1948, and the designers have stated that their studies and design were carried out with full knowledge and use of all prior and current data reports.[4]

In addition to the project planning, the writer wishes to refer to the economic and financial aspects as described in the paper:

"The project will be financed with funds available from oil royalties . . ."; (see headings, "History of the Project" and "Purpose of Project").

"The primary purpose of the project is to provide a dependable industrial and domestic water supply for the rapidly growing city of Tehran. The development of water supply at Karadj will also provide some benefits to the flood control area. It was found economical in connection with the development of water supply to install a hydro-electric plant to provide peaking capability for the Tehran electrical system."

"The total annual charges, using an assumed interest rate of $4\frac{1}{2}\%$ are estimated to be equivalent to $3,700,000. Just over 50% of this will be recovered from electrical revenues, and the remainder will be charged against water supply. When fully utilized, this will provide water at $0.01 per cu m and this is deemed to be reasonable under the circumstances. Benefits to result from irrigation water supply and flood control have not been estimated for the purpose of justifying the project."

With this rather brief background, the authors have devoted most of their paper to the technical design aspects. Because their paper makes only this brief reference to the economic and financial aspects of the project and to the basic problems of its planning, the writer wishes to offer some additional facts which should be of general interest to all engineers.[5]

Irrigation.—No mention is made by the authors of design requirements to supply 250,000,000 cu m per yr of free controlled irrigation.

Initially, twenty owners of 39 village estates covering 22,240 acres in the vicinity of Karadj will be supplied 75,000,000 cu m per yr of free controlled release irrigation. Later, with construction of a 27-mile aqueduct at public expense, kindred ownership of an additional 112 village estates comprising an added 56,600 acres will have available 180,000,000 cu m of free controlled release irrigation. Thus, some fifty related owners will receive free an average of 3,156 cu m for each of 78,900 acres, or 2.3 plus acre-ft of water per yr.

This generous supply of free controlled irrigation is lightly referred to in the paper as "some benefits to the flood control of the area."

Harza Report, 1955.—The Harza Engineering Company was engaged June 20, 1955 to digest all available data pertaining to the Karadj project and to supply an "office study" of economic feasibility. On August 8, 1955, Harza delivered its affirmative report to the United States Mission to Iran in Tehran. The report said:

> "The Karadj Dam Project will satisfy a great human need. A potable water supply is essential to the health, growth, and advancement of a metropolitan area. Potable water is dependent upon an assured supply of good quality and an adequate distribution system. Steps are being taken by the Tehran water supply, with the consulting service of Sir Alexander Gibbs and Partners, to provide an adequate distribution system. Karadj Dam is needed urgently to give the assured supply."

A $50,000,000 Karadj Dam with a minimum annual controlled storage capacity of 320,000,000 cu m was "needed urgently" to assure a potable water supply to Tehran, which could use no more than 40,000,000 cu m of annual controlled storage until 1980!

Peaking Power.—The revenue from peaking power is estimated at $1,850,000 for an annual delivery of 149,000,000 kwh. This is an annual delivery of 1,862 kwh per kw for 80,000 kw of peaking capacity, or 5 hr a day use.

The kilowatt-hours available for delivery to Tehran for true peaking will be somewhat under 60,000,000 kwh per yr, 89,000,000 kwh short of the annual deliveries given for revenue purposes. This excess of kilowatt-hour deliveries is, in fact, "dump" seasonal energy, varying from year to year. It may or may not be economical for the Tehran Power System to absorb this dump energy, and certainly not at peaking rates. Fuel replacement rates, the criteria for dump energy, should be less than 4 mills per kwh, in lieu of the 1.2 cents average rate per kwh cost recovery justification for the Karadj construction.

Iran has one of the world's largest oil reserves, a major field within 60 miles of Tehran, and pipeline transmission from all producing sources.

Tehran Water Supply.—The purported purpose for the Karadj Project given by the authors—potable water storage for Tehran—could have been provided for $6,000,000; Karadj is costing $50,000,000, as a minimum.

Tehran's maximum potable water storage requirement to 1980 is 40,000,000 cu m. The Karadj design total storage capacity is 205,000,000 cu m, with a minimum annual controlled release capacity of well over 320,000,000 cu m, eight times Tehran's 1980 storage needs.

Annual payments by Tehran of $1,850,000 for 40,000,000 cu m of storage capacity appears somewhat excessive. This is a total cost recovery in about 3 yr for the fairly apportioned dam costs for Tehran's maximum storage needs.

Karadj Revenues, 1970.—The authors report estimated revenues, as:

Annual Deliveries	Annual Revenues
Hydro-Electric Power—Peaking—149,000,000 kwh	$1,850,000
Tehran Water Supply Service—Cubic Meters? ?	1,850,000
Controlled Irrigation Service—Cubic Meters? ?	0
Total	$3,700,000

This would indicate delivery of 149,000,000 kwh per yr for peaking power, or 400,000 kwh daily, 5 hr a day of 80,000 kw.

Controlled irrigation deliveries of 131,000,000 cu m per yr, with revenues of $336,000 included by Harza in arriving at the affirmative 1955 economic justification, were not included in the 1960 report; their inclusion in the 1955 analysis was contrary to the established and reported facts.

The Harza 1960 total revenue estimate of $3,700,000 is $1,353,000 more than the Harza 1955 estimates of $2,347,000 for electrical power and potable water deliveries alone. The quantities delivered for each of the two services cannot have increased enough to warrant the reported dollar increases, except through unit rate increases of appreciable magnitude.

Have these rate increases, in fact, been accepted by the Tehran Power Authority and the Tehran Water Supply Service? If so, for what reasons?

The question also arises that if total annual revenues of $2,713,000 in 1955 were more than sufficient to yield affirmative economic justification for a Karadj Dam project to cost an estimated $54,000,000 to $64,000,000, for what reason are revenues of $3,700,000 necessary in 1961 to support a $50,000,000 construction?

Can it be that the excess charges collected from the Tehran water and electric consumers will be used to build, with continuing design and construction, the 100 miles of main aqueduct and laterals necessary to deliver to the village estate owners the 250,000,000 cu m of controlled free irrigation included in the Karadj construction design?

The design, and the current construction, does include the availability of 250,000,000 cu m of free controlled irrigation release, in addition to the water for seasonal power generation and continuous potable water supply to Tehran.

It is accepted that the Karadj design is adequate and construction competent, given the physical limitations of the Karadj basin, and the hazards of the site.

Conclusions.—The official record of the Karadj Project seems to cloak it in a respectability to which it is not entitled. The factual history of this operation as extracted from the strangely distorted record shows that:

The Karadj Project as designed and built is a $50,000,000 gift from the United States to absentee village owners of virtually unlimited quantities of free regulated irrigation for their already choice farm lands.

In a water-hungry area where the mean rainfall is 9 to 10 in. per year, this free supply of controlled irrigation will fantastically increase the land values and incomes of these few estate owners.

The water and power users of Tehran will be grossly overcharged to give Karadj the illusion of economic and financial virtue.

Official.—Official documents and testimony state United States Karadj financing was limited to $1,400,000, of which less than $750,000 was spent for initial surveys, consultants, reports, design, engineering, and minor construction.

The reported estimated cost to completion of Karadj was $17,000,000, which would provide adequate water and power for Tehran and its suburbs, and irrigation for over 9,000 hectares (22,240 acres) of land in the vicinity of Karadj.

The now completed Karadj Hydroelectric Project will yield essentially the same controlled water storage and releases as the original designs of January 1950 used as the basis of the Karadj promotion. Actual initial estimated cost— $38,000,000.

The Record.—United States officials guaranteed Iran in 1953 that the United States would finance the Karadj Project to completion, and immediately:

Assembled appropriated funds for the Karadj construction—	$6,500,000
Spent from this account for Karadj construction	3,500,000
Balance directly uncommitted	$3,000,000

Suspension.—The Karadj promotion by United States officials came under external scrutiny in late 1954, and responsible Iranians opposed to this wastage of Foreign Aid funds were temporarily effective. The $3,000,000 remaining in the Karadj account were withdrawn, merged into general accounts, and their ultimate use cannot be traced.

The Karadj construction was suspended, as Iranian funds officially reported as available for the Karadj project were non-existent.

Revival.—By mid-1957 political and financial conditions again became propitious for reactivation of the Karadj promotion, and the $46,500,000 of United States appropriated funds to fulfill the 1953 official pledges have been passed through a series of obscuring accounts to finance Karadj to completion.

Official documents and testimony state that all United States guidance and funds were withdrawn from the Karadj promotion in early 1955, and that subsequent Karadj developments have come solely from Iranian volition.

STRESS ANALYSIS AND SPECIAL PROBLEMS
OF PRESTRESSED CONCRETE DAMS[a]

Discussion by D. Hugh Trollope and Ian K. Lee; and Robert E. White

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D. HUGH TROLLOPE,[5] M. ASCE, IAN K. LEE.[6]—The topic of cable pre-stressing method applied to mass concrete dams, as presented by the authors, is one that has aroused considerable interest in recent years. In the introduction to the present paper a valuable summary of the developments in this field is given.

During the past year (1961) or so, the writers have been privileged to participate with the Hydro-Electric Commission, Tasmania, in a series of tests on relatively large scale models of the Catagunya Dam referred to in the paper. The models, essentially right-triangular in cross section (base 4.75 ft, height 6 ft) and 10-ft wide were founded on specially selected rock foundations. Provision was made for varying the load transmitted by the stressed cables, and the water load was simulated by a suitably placed horizontal jacking system (Fig. 25). The models were designed and the tests conducted by the staff of the Hydro-Electric Commission under the supervision of J. K. Wilkins, Engineer for Civil Design.

The writers were consulted concerning the possibility of measuring the contact pressure distribution between model and foundation.

It is extremely difficult, if not impossible with available techniques, to measure internal strains (stresses) with concrete masses of the size concerned in these models. Hence, it was concluded that the only practicable approach was to attempt to measure the pressures on the contact plane.

From previous experience, the writers (37) had concluded that with a sufficiently sensitive pressure cell, satisfactory measurements of boundary pressures could be obtained, provided a layer of compact sand of thickness not less than the effective diameter of the cell was placed immediately over the cells. A suitable cell had been designed, constructed, and extensively calibrated in the laboratory (36) and as the diameter of these cells was 0.75 in. with a pressure responsive diaphragm characteristic of 2.8 by 10^{-6} in. per lb per in.^{2}, it was decided that a layer of sand approximately 1-in. thick would be adequate to permit measurement of the vertical stresses across the base.

The pressure cells were, therefore, installed in the foundation and covered by a 1-in. layer of carefully placed compacted sand. The concrete model was then carefully poured on this sand layer (Fig. 25).

The decision to introduce the sand layer in the model tests entailed the two following significant considerations with respect to the prototype: (a) the shear

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resistance across the sand—concrete interface in the model would not be adequate or representative; and (b) the physical characteristics of the sand might be foreign to the concept of a rock continuum underlying the actual dam.

Adequate horizontal shear resistance was readily provided by a suitable toe support but the question of the sand interference is not as easy to answer. It is the writers' opinion, however, that the compact sand bed is a closer approximation of the real prototype conditions than any other available system.

Virtually all rock masses in nature are fissured or jointed in three dimensions or both, and this is particularly apparent on the site of the Catagunya Dam (38). Therefore, it is erroneous to consider such a rock mass as a continuum, rather it should be regarded as a compact granular mass the behavior of which is governed by interparticle reaction, and intra-particle strength is
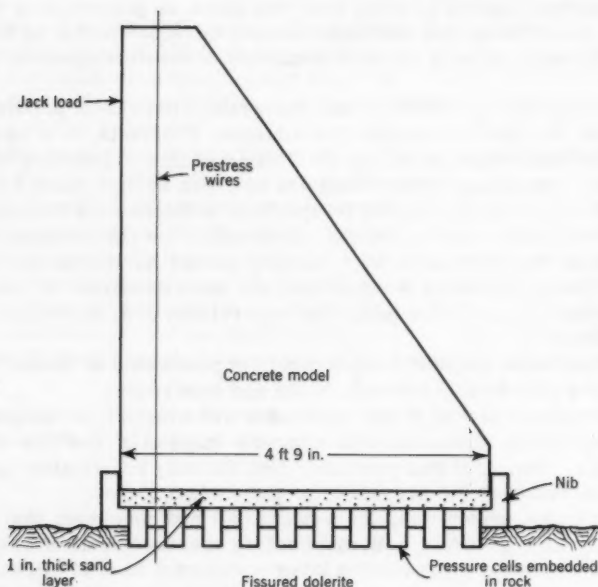

FIG. 25.—CATAGUNYA DAM MODEL EXPERIMENTAL AR-
                        RANGEMENT

of minor consequence. Unless the particle size is of the same order as the width of the dam (100 ft cube in the case of Catagunya Dam) the rock foundation cannot be considered as a continuum.

Another aspect in support of the sand bed is that, as remarked by the authors, it is customary to assume that no tensile stresses are permissible in the foundation, and the sand layer guarantees this condition.

With the experimental arrangement previously described, consistent results were obtained and some typical distributions are shown in Fig. 26. The applied loads are, in each case, reduced to the equivalent eccentricity e.
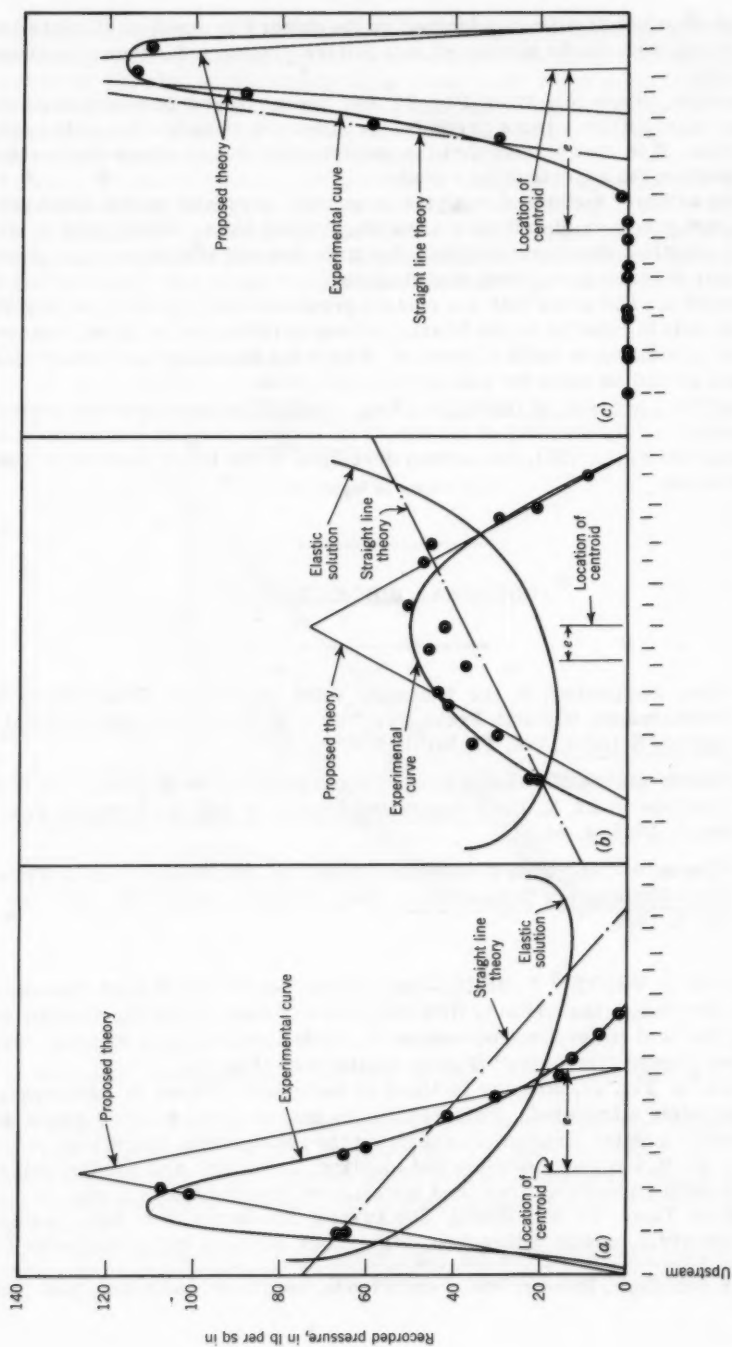
FIG. 26.—CATAGUNYA DAM MODEL CONTACT PRESSURE DISTRIBUTION

The theoretical distribution derived by the writers is based on ultimate load conditions as well as the measured sub-failure pressure diagrams measured in the tests.

Meanwhile, it is clear from Fig. 26 that the measured pressure distributions are different from those predicted by either the straight-line or the elastic theories. It is also evident that the straight-line theory shows the greatest deviation from the experimental results.

As the authors' method of analysis is entirely governed by the assumption of a contact pressure distribution according to this latter theory, and it also assumes elastic continuum conditions for both dam and foundation, it is doubtful whether it has any practical significance.

It should also be noted that the contact pressure distribution is of significance not only in relation to the internal stress problem but is of serious consequence in relation to uplift allowance. Where the base pressure is zero then allowance should be made for full uplift in this zone.

The writers are also of the opinion that, although, according to the authors, their analysis of the stability of the dam (Fig. 19) gives results similar to those of Wilkins and Fidler (38), the method developed by the latter workers is more soundly based.

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ROBERT E. WHITE,[7] F. ASCE.—Borrowing some of anchorage features of prestressed dams, the writer's firm has recently been installing sheeting and bracing for land cofferdams by means of prestressed anchors to rock; these have been termed "tie-backs" (Patent applied for) (Fig. 27).

As seen in Fig. 27, interior inclined or horizontal braces in compression are completely eliminated. This results in unobstructed working space and consequently greater economy in executing the construction operations: excavation of earth and rock, drilling and blasting, formwork and reinforcing for concrete walls, slabs and floor, and installation of waterproofing, etc.

Loads on Ties.—To date (1961), 266 Pretest Tie-backs have been installed on four projects, one in Milwaukee, Wis. (rock stratum horizontally-bedded

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seamy soft limestone), two in New York City (mico schist) (40), (41); and one in Rochester, N. Y. (Niagara limestone). Design loads on the Milwaukee ties ranged from 123 kips to 154 kips in direct tension; on the New York City work, the loads were 80 kips to 230 kips; and in Rochester, up to 120 kips.

Holes in Rock.—Diameter of the drilled holes is usually $3\frac{1}{2}$ in. These holes are drilled by percussion machines (crawler drills).

Types of Tendons.—The writer prefers the use of the word "tendon" to "cable" because it is more generic. On the Milwaukee job, the first to be done, button-headed (BBRV) wires 0.25 in. ϕ were used. These had the drawback that they had to be made up in fixed lengths in advance; because rock surfaces may not be level, this poses the problem of what length to fabricate, or else
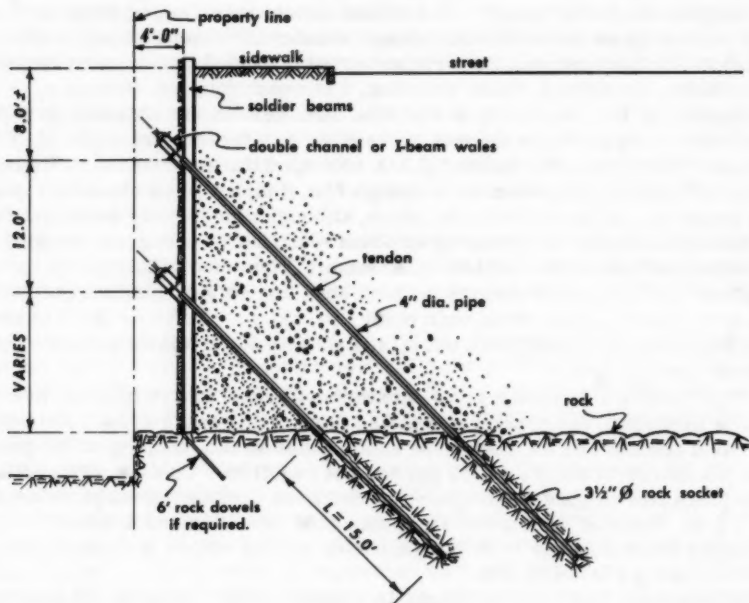

FIG. 27.—TIE-BACKS

the necessity of drilling deeper sockets in rock to accommodate overlength tendons.

On subsequent work, stressteel rods up to $1\frac{1}{4}$ in. have been used where lighter loads (up to 180 kips) are encountered. For heavier loads, 7-wire $\frac{1}{2}$ in. diameter strands or stress-relieved steel are used. Minimum guaranteed ultimate strength of the Stressteel rods is 145,000 psi; of the 7-wire strands it is 259,000 psi.

Design of Anchorage.—In accordance with the principles presented by M. Coyne, the weight of rock mass, in the shape of inverted overlapping cones en-

gaged by the tie-backs, is the force that is relied on to anchor the sheeting. It is assumed that the rock will shear at an angle of 45° to the pull of the tie.

It has been found that holes of 10-ft to 16-ft lengths are required to mobilize sufficient weight of rock for safe anchorage. These lengths are amply sufficient to keep the bond stresses between the tendons and the grout, and between the grout and the rock to a low value. Anchorages of the button-headed wires and of the Stressteel rods are further assisted by the provision of a 3-in. diameter plate at the bottom end. Because of the lengths of the holes demanded by the foregoing design criterion, bond stresses, when using $\frac{1}{2}$ in. diameter 7-wire strands, have not exceeded 80 psi between strands and grouts and have not exceeded 94 psi between grout and rock. These are felt to be conservative values based on past tests.

Grout Composition.—Nevertheless, the grout function is so important that the highest quality is sought for and numerous experiments have been made with various grout materials including: standard Portland cement, high-early strength Portland cement, neat cement grouts, sanded grouts, admixtures for retardation, densifying, water reducing, and expansion, etc.

Finally, it has been determined that best results are obtained by using a neat high-early strength cement grout with a water-cement ratio of 5 U. S. gal per 94 lb sack (w/c ratio of 0.44). No admixtures are used. Their possible benefits do not seem to outweigh the complications of correct on-the-job measuring of the ingredients, which, although simple, have led to mistakes.

Colloidal Mixing and Pumping of Grout.—Mixing of the grout is done in a gasoline powered Colcrete 56M roller mixer. The drum of this mixer revolves at about 1,800 rpm and through its high shearing action completely separates the fine cement grains from each other, so that no clumps or flocs of cement remain, however, complete wetting and subsequent complete hydration of the cement are attained.

This "colloidal" mixing gives maximum fluidity with minimum water and this is important in securing complete filling of all space around and between the steel tendons and the rock. The fluidity aids in the pumping of the grout so that pressures in excess of 100 psi are not required. To date, little difficulty with plugs and stoppages have been encountered although grout pipes as small as 1/2 in. diameter are used. The pump used is a Colmono 4, which is a progressive helical cavity type (Moyno) with rubber stator and stainless steel rotor—capacity is 1 1/2 cfm.

Mixing time in the roller mixer is approximately 15 sec to 20 sec so that sufficient high hourly production for such a small unit (1 sack of cement per batch) is no problem.

In three days, the previous neat grout reaches a strength in excess of 4,000 psi based on compression tests on 6 in. diameter by 12 in. high cylinders.

It frequently happens that ground water seeps into the drilled holes. Causes of this may be an imperfect seal of the casing to the rock, or water-bearing fissures and seams in the rock itself. No attempt is made to seal off infiltration of water. The ground water is allowed to seek its own level in the hole. However, the hole is cleaned of mud or silt by connecting up the grout pipe to a clean water supply and the flushing water until the effluent runs clear. Then the grout is introduced at the bottom of the hole and is tremied into place. The principle here is to avoid washing out and weakening the grout mixture by ground water seeping in against an unbalanced head.

This procedure is contrary to what the authors describe as "the commonly adopted procedure" namely "to feed the cables into grout previously deposited at the bottom to the anchorage hole." While this may be true, the writer believes this to be a wrong procedure that leads to frequent pulling-out of the tendons under test. The feeding of the tendon into the hole may involve pulling-up at intervals to work the cable down to the bottom of the socket as the rock sides of the hole are never absolutely smooth. On the contrary, they are often quite rough (which is all to the good considering the necessity of good bond).

This working of the cable stirs up the grout and can cause dilution from ground water that is usually present in the fissures, cracks, and seams known to exist even in the soundest rock.

Sometimes attempts are made on some dam jobs to seal off all leakage by pressure grouting of the socket and subsequent re-drilling of the hole. The assumption is that a perfect seal can be obtained. From writings on the subject, it seems evident that perfect seals are never obtained and an excessive number of holes must be repeatedly regrouted and redrilled because of the pulling out of the tendons under test.

Post-tensioning.—Since this high strength is gained in such a short period as 3 days, the post-tensioning of the tendons can follow quite promptly and conveniently after the grouting operation and no delays are encountered on this score. On occasion where a tight schedule demands it, ties have been post-tensioned 48 hr after grouting.

The post-tensioning of the tendons is customarily done to a load that is equal to the yield point of the steel. The reason for this is to give the maximum possible "Pretest" of the tie-back system without damaging the tendons themselves.

As an extreme test of the grout, 12-1/2 in. diameter 7-wire strands were inserted into a 3 1/2 in. diameter vertical hole drilled 20 ft deep into rock. The bottom 9 ft were grouted in. Exactly 72 hr later, a load was put on the strands of 388,000 lb. At this point the wires started breaking and the test was discontinued. No pulling out of the cables from the grout, nor the grout from the rock could be observed. The bond between the grout and the rock was thus tested without failure to a stress of 327 psi.

As a final word, the writer would caution against embedding wires in rock sockets by means of grouts containing so-called expanding agents if these agents depend on the creation of expansion by a salt acting on iron fillings to form a rust. The salt will also attack the wires and may critically reduce their area.

Although the preceding discussion is based mainly on the writer's experience with prestressed ties in rock on other than dam work, it is hoped that his experience may prove to be helpful in the latter field.

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POWER FROM THE SEA[a]

Discussion by George A. Whetstone

GEORGE A. WHETSTONE,[5] F. ASCE.—Hassan has made a welcome contribution to a subject on which little has been published in readily available sources. His reference to the unlocated paper by M. R. Bigarre typifies the status of the literature in the field of hydro-solar power. Thus, when L. Vadot (23)[6] sketched a possible development of the Red Sea he based his figures on an unpublished study of M. Bard. Authors of earlier papers on the Dead Sea, namely E. Imbeaux (5), E. E. Shalowitz (16), and others, disagree among themselves in crediting the original inspiration for that project to Paul Simon (1903) (1), to Herzl ("about fifty years ago" in a 1947 paper) (16), and to Hjorth (2) who, in turn, cited Biblical inspiration. No power development seems to have been contemplated by Roudaire and De Lesseps in their project for creating an interior sea in Algeria in the 1870's. Later references to this project often refer to power heads but without clear indication as to when or by whom this modification of the original venture first occurred.

The Red Sea is one of the most promising of some twenty sites that have received consideration for power development based on the evaporation of tailwater. The salient features of many of the others are included in Vadot's paper (23) and in other entries in the attached bibliography on hydro-solar power.

The cost figures cited by Hassan ($33,000,000,000 total, $6,717,000,000 for maximum debt, and so forth), certainly make far different reading than did the proposals by Pierre Gandillon, published in 1931 (9-11), for a step-by-step development of the Goubbet Kharab, Lac Hallol, and Lac Assal with much of the preliminary work being done in France and then towed on a barge to French Somaliland for installation. For his Lake Maracaibo project he did quote a total cost of $2,000,000,000, but this was in francs at a time when they were exchanged at the rate of 25 francs for one dollar.

As Vadot has shown, for a project such as that of the Red Sea where a period of years must elapse while evaporation is creating the power head, it is possible to avoid much of the immobilization of capital by using an impervious curtain and then building the dam slowly as the increasing head requires greater strength.

It might also be of interest to investigate the feasibility of "pumped unstorage" (34) by considering a combination of the Red Sea project with those of Lac Assalé (23), of the Danakil Depression (14, 15), and of Lac Assal (9, 24, 25) all of which are within easy transmission distance of the dam sites proposed

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by Hassan in the straits of Bab el Mandab. For a land-locked depression excess power beyond that of the ultimate equilibrium wherein inflow balances evaporation may be obtained while the basin is filling to its ultimate level. Perhaps even some of the turbines destined for use in the Red Sea could be employed in temporary housing near the bottoms of the depressions. While the effect on the head in the Red Sea would, of course, be small due to the large area involved, the availability of power during the 2 yr of zero inflow contemplated might well facilitate dam construction and the establishment of electrochemical and other industries that are to become the ultimate customers for the power developed.

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OROVILLE DAM AND APPURTENANT FEATURES[a]

Discussion by Erik Rettig

ERIK RETTIG,[4] F. ASCE.—The design of the Oroville dam contains features of great interest, and it is hoped that the planning and design of this important structure will be more fully covered in later publications. It is realized that many details still remain to be worked out and revised before a final design can be presented. For many similar structures expedient and improving revisions often create a continual process that may last until near completion of construction, but in this case, well before then a paper with more extensive information on the main features and special details of the design of the dam would make a valuable contribution to professional records and knowledge.

It would be interesting at this time if the authors will give additional information on the studies and reasoning connected with the design of the core and toe blocks. Judged on the basis of the limited information given, and experience from dealing with similar seasonal flood overflow cases, the design appears basically good, as one of several possible solutions to fill-containment problems, but it occasions some thoughts, strengthened by admittedly superficial field inspection of the proposed fill material. The shape, size and weight characteristics of the material indicated relatively high mobility in moving water. Theoretical and model test studies can provide good guidance on this aspect, but some extra caution appears advisable. Assuming that it is the intention to fill the space between the blocks approximately to a straight plane between their top levels, and that the top elevations indicated, or later revised, for the core and toe blocks are carried horizontally across the temporary flood channel, the top elevation of the toe block should probably, if at all practically possible, be higher than indicated in Fig. 2, relative to the top of the core block, unless other special provisions are planned to prevent or minimize transportation and loss of material through rolling and saltation movements. In practice, heavy debris, submerged tree stumps, logs and boulders carried along by flood water may radially upset and continually change the flow pattern across the temporary channel bottom, with concentration of higher than average velocities easily resulting in excessive removal of fill material. Even if it is intended to keep an area low immediately upstream of the core block, acting as a rock and debris trap, changing the "sill" elevations, and with this the slope of the channel bottom, as suggested should be beneficial. It may result in some undesirable material being deposited over the placed fill material, but such material can generally be removed easier and at less cost than replacing lost proper fill.

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[a] July 1961, by W. G. Schulz, D. P. Thayer, and J. J. Doody (Proc. Paper 2852).
[4] Cons. Engr., Belvedere—Tiburon, Calif.

It would also be interesting to learn what provisions may be contemplated to intercept and handle possible seepage along the top surface of the core block. One or more plastic-elastic membranes embedded in the concrete of the core block and extending to some height into the core material is offered as a suggestion. Work has been carried on for some time on the development of this principle to supplement, and in some cases take the place of, an "impermeable" earth core in embankment dams and similar structures, and it is being considered in some actual designs.

MODEL AND PROTOTYPE RESEARCH ON FISH LADDERS[a]

Discussion by C. H. Clay

C. H. CLAY.[11]—The author has covered in general terms much of the latest work on fish facilities on the Columbia River. The high value of the Columbia River salmon runs and the large scale of the fish facilities required on that river has warranted extremely detailed investigations that are of particular interest to engineers involved with design of fish facilities at other locations. These investigations have been followed closely in Canada in order to take advantage of the results where applicable; however, Canadian interpretation of the results differs in some respects from those presented by the author. Although this difference may seem small it involves principles of design that are considered worthy of comment herein.

Fishway Capacity Studies.—The author states that "during one specific experiment salmon were passed through a 1 on 16 slope fishway only 4 ft wide at a rate of 3,000 fish per hr without any indication that passage capacity had been reached." The Fish and Wildlife Service publication[12] covering the tests done in 1956 lists the results of several capacity tests. Among these was one performed on Sept. 7, 1956, during which the capacity was tested at a rate of approximately 3,000 fish per hr. No comment is made in this report on whether the fishway capacity had been reached, but a further report was published by the same agency a short time later analyzing the results.[13] In the later report, it is clearly demonstrated that the rate of fish leaving the fishway decreased with increased numbers present in the fishway during the test cited, and the statement is made "we conclude that movement was hindered by crowding." It is felt, therefore, that the author of the present paper should have noted that other tests were made with different results from those quoted.

Without any knowledge of the test quoted by the author, the writer is inclined to rely on the results published by the Fish and Wildlife Service that were carefully documented. Incidentally, the latter have been substantiated by observations in fishways in Canada, where capacity has apparently been reached when fish have been observed in fishways in concentrations comparable to those cited.

Fish Ladder Length and Slope.—In this section, the author states that "the rate of fish movement in a fish ladder is independent of the number of fish present." The same comments as the preceding apply to this statement. In a well-documented test the Fish and Wildlife Service biologists demonstrated

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a July 1961, by Berton M. MacLean (Proc. Paper 2856).

11 Ch. Engr., Pacific Area, Dept. of Fisheries, Vancouver, B. C.

12 "Fishway Capacity Experiment, 1956," by Carl H. Elling and Howard L. Raymond, U. S. Fish & Wildlife Service, Special Sect. Report. No. 299, May, 1959, pp. 12, 20-26.

13 "The Problem of Fishway Capacity," by Robert H. Lander, U. S. Fish and Wildlife Service, Special Sect. Report. No. 301, May, 1959, p. 4.

clearly that the rate of fish leaving the fishway decreased with the number present. Therefore, the author's statement would appear to be not a valid generalization.

Earlier in this section the author mentions monetary savings that would be realized by building a fishway at a slope of 1 on 8 rather than 1 on 16. There is no doubt that a fishway constructed at a slope of 1 on 8 would be less costly than one constructed at a slope of 1 on 16, provided they were of equal width. It has, however, long been a fishway design principle both on the Columbia River and in Canada, that fishway capacity is related primarily to the volume of the fishway rather than to any one dimension such as length. The soundness of this approach has been borne out by the results of the capacity tests cited by the author and the analyses conducted by Fish and Wildlife Service biologists. It has also been further substantiated by observations in Canada.

It is, therefore, misleading to imply that a fishway with a 1 on 8 slope can do the same job as a fishway of equal width with a 1 on 16 slope, since the 1 on 8 slope fishway will have only about half the capacity of the 1 on 16. There are many successful fishways on the Pacific Coast of Canada and the United States which have been constructed with slopes varying from 1 on 6 to 1 on 20. While it is reassuring to learn from the tests cited by the author that the energy requirements for ascent of the fishways with the steeper slopes in this range are not excessive, it cannot be deduced that large savings in construction will result from building fishways at the steeper slopes.

From an analysis of the United States Fish and Wildlife Service tests cited it is evident that fishway capacity is related to the rate of ascent of the fish and the instantaneous volume requirements of the fish at any particular location. Assuming a uniform head difference between pools and reasonably favourable hydraulic conditions within the pools, capacity can be directly related to the volume of the pools. The shape of the pool (width, length, and depth) and the fishway slope are (within limits) only important in relation to meeting hydraulic and structural requirements. Some economies might be effected by varying these dimensions for each new fishway, but these economies would be relatively minor compared to those implied by the author.

It is felt it is important to cite these principles in commenting on the author's otherwise excellent paper. Much useful information has resulted from the recent research work on the Columbia River and the results are well worth bringing to the attention of engineers in many countries.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU); and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

VOLUME 86 (1960)

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653 (HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)^C, 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)^C, 2665(HY9)^C, 2666(SA6)^C, 2667(ST11)^C.

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675(EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)^C, 2692(ST12), 2693 (ST12), 2694(HW4)^C, 2695(IR4)^C, 2696(SM6)^C, 2697(ST12)^C.

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JANUARY: 2698(PP1), 2699(PP1), 2700(HY1), 2701(SA1), 2702(SU1), 2703(ST1), 2704(ST1), 2705(SU1), 2706 (HY1), 2707(HY1), 2708(HY1), 2709(PO1), 2710(HY1), 2711(HY1), 2712(ST1), 2713(HY1), 2714(PO1), 2715 (ST1), 2716(HY1), 2717(SA1), 2718(SA1), 2719(SU1)^C, 2720(SA1)^C, 2721(ST1), 2722(PP1)^C, 2723(PO1)^C, 2724(HY1)^C, 2725(ST1)^C.

FEBRUARY: 2726(WW1), 2727(EM1), 2728(EM1), 2729(WW1), 2730(WW1), 2731(EM1), 2732(SM1), 2733(WW1), 2734(SM1), 2735(EM1), 2736(EM1), 2737(PL1), 2738(PL1), 2739(PL1), 2740(EM1), 2741(EM1), 2742(ST2), 2743(EM1), 2744(WW1), 2745(WW1), 2746(SM1), 2747(WW1), 2748(EM1), 2749(WW1), 2750(WW1)^C, 2751 (EM1)^C, 2752(SM1)^C, 2753(PL1)^C, 2754(ST2)^C, 2755(PL1).

MARCH: 2756(HY2), 2757(IR1), 2758(AT1), 2759(CO1), 2760(HY2), 2761(IR1), 2762(IR1), 2763(HY2), 2764 (ST3), 2765(HY2), 2766(HW1), 2767(SA2), 2768(CO1), 2769(IR1), 2770(HY2), 2771(SA2), 2772(HY2), 2773 (CO1), 2774(AT1), 2775(IR1), 2776(HY2), 2777(HY2), 2778(SA2), 2779(ST3), 2780(HY2), 2781(HY2)^C, 2782 (HW1)^C, 2783(SA2)^C, 2784(CO1), 2785(CO1)^C, 2786(IR1)^C, 2787(ST3)^C, 2788(AT1)^C, 2789(HW1).

APRIL: 2790(EM2), 2791(SM2), 2792(SM2), 2793(SM2), 2794(SM2), 2795(SM2), 2796(SM2), 2797(SM2), 2798 (EM2), 2799(EM2), 2800(EM2), 2801(EM2), 2802(ST4), 2803(EM2)^C, 2804(SM2)^C, 2805(ST4)^C.

MAY: 2806(SA3), 2807(WW2), 2808(HY3), 2809(WW2), 2810(HY3), 2811(WW2), 2812(HY3), 2813(WW2), 2814 (HY3), 2815(WW2), 2816(HY3), 2817(HY3), 2818(SA3), 2819(WW2), 2820(SA3), 2821(WW2), 2822(WW2)^C, 2823(HY3), 2824(SA3), 2825(HY3), 2826(SA3)^C, 2827(HY3)^C.

JUNE: 2828(SM3), 2829(EM3), 2830(EM3), 2831(IR2), 2832(SM3), 2833(HW2), 2834(IR2), 2835(EM3), 2836 (IR2), 2837(IR2), 2838(SM3), 2839(SM3)^C, 2840(IR2)^C, 2841(HW2)^C, 2842(EM3)^C, 2843(ST5), 2844(ST5), 2845(ST5), 2846(ST5)^C.

JULY: 2847(PO2), 2848(SU2), 2849(HY4), 2850(PO2), 2851(HY4), 2852(PO2), 2853(SU2), 2854(HY4), 2855 (PO2), 2856(PO2), 2857(PO2), 2858(SA4), 2859(SU2), 2860(SA4), 2861(PO2), 2862(SA4), 2863(HY4), 2864 (HY4), 2865(HY4), 2866(HY4), 2867(HY4), 2868(PO2)^C, 2869(SA4)^C, 2870(SU2)^C, 2871(HY4), 2872(HY4)^C, 2873(SU2), 2874(SA4).

AUGUST: 2875(WW3), 2876(WW3), 2877(WW3), 2878(SM4), 2879(ST6), 2880(EM4), 2881(SM4), 2882(EM4), 2883(WW3), 2884(EM4), 2885(SM4), 2886(WW3), 2887(EM4), 2888(WW3), 2889(AT2), 2890(AT2), 2891 (AT2), 2892(AT2), 2893(AT2), 2894(AT2), 2895(AT2), 2896(AT2), 2897(AT2), 2898(AT2), 2899(AT2), 2900 (AT2), 2901(AT2), 2902(SM4), 2903(ST6), 2904(ST6), 2905(SM4), 2906(ST6), 2907(EM4), 2908(ST6), 2909 (EM4), 2910(ST6), 2911(EM4), 2912(SM4), 2913(ST6), 2914(WW3)^C, 2915(ST6)^C, 2916(EM4)^C, 2917(SM4)^C.

SEPTEMBER: 2918(SA5)^C, 2919(HW3)^C, 2920(HY5)^C, 2921(SA5), 2922(PL2), 2923(IR3), 2924(HY5), 2925 (HY5), 2926(CP1), 2927(HY5), 2928(IR3), 2929(IR3), 2930(HY5), 2931(CP1), 2932(PL2), 2933(HY5), 2934 (HY5), 2935(HY5), 2936(HY5), 2937(HW3), 2938(CP1), 2939(PL2), 2940(SA5), 2941(SA5), 2942(SA5), 2943 (HY5), 2944(PL2)^C, 2945(CP1)^C, 2946(IR3), 2947(HW3), 2948(IR3), 2949(IR3)^C.

OCTOBER: 2950(PP2), 2951(PP2), 2952(PP2), 2953(ST7), 2954(SM5), 2955(ST7), 2956(ST7), 2957(ST7), 2958 (ST7), 2959(SM5), 2960(EM5), 2961(EM5), 2962(ST7), 2963(ST7), 2964(EM5), 2965(SM5), 2966(SM5), 2967 (ST7), 2968(ST7), 2969(ST7), 2970(ST7), 2971(SM5), 2972(SM5)^C, 2973(EM5), 2974(ST7)^C, 2975(PP2).

NOVEMBER: 2976(HY6), 2977(HY6), 2978(HY6), 2979(PO3), 2980(WW4), 2981(HY6), 2982(HY6), 2983 (HY6), 2984(HY6), 2985(SA6), 2986(SA6), 2987(WW4), 2988(WW4), 2989(WW4), 2990(HY6), 2991 (WW4), 2992(HY6), 2993(PO3), 2994(WW4), 2995(PO3), 2996(HY6), 2997(SA6), 2998(PO3), 2999 (PO3)^C, 3000(HY6)^C, 3001(SA6), 3002(SA6), 3003(SA6)^C, 3004(WW4)^C.

c. Discussion of several papers, grouped by divisions.